ATTACHMENT A

Geotechnical Investigation Report

for

Mast Park Improvements Project
CIP 2008-53

Geotechnical Investigation Report

Mast Park Improvements
9125 Carlton Hills Blvd
Santee, California

<u>Prepared for:</u>
Dokken Engineering
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August 31, 2016
Project No. 160392.2

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August 31, 2016
Project No. 160392.2

Mr. Mark Tarrall
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San Diego, California 92123

Subject: Geotechnical Investigation Report
Mast Park Improvements
9125 Carlton Hills Blvd
Santee, California

Dear Mr. Tarrall,

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the Mast Park improvements project in the City of Santee, California. The purpose of this investigation has been to evaluate the subsurface conditions at the site and to provide geotechnical engineering recommendations for the proposed project.

Please note that the recommendations presented within the report are based on assumptions stated herein. Should conditions encountered during development differ from those assumed in our analyses, or should the proposed development change, our recommendations may need to be modified accordingly. This report should be submitted to the appropriate authorities as part of the process of obtaining development permits for the project.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
TWINING, INC.

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1. INTRODUCTION

This report presents the results of the geotechnical evaluation performed by Twining, Inc. (Twining) for the proposed Mast Park improvements project, located at 9125 Carlton Hills Blvd in the City of Santee, California as shown in Figure 1, Project Location Map. The purpose of this study has been to evaluate subsurface conditions and provide geotechnical engineering recommendations relative to the design and construction of the proposed project. The objectives of this study were to evaluate the subsurface conditions of the site, and to provide geotechnical recommendations for the design and construction of the proposed improvements, including recommendations for foundations and earthwork.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The project site is located on the north side of the San Diego River, east of Carlton Hills Boulevard and Carlton Oaks Plaza, west of the Mission Creek neighborhood and south of Carlton Country Club Villas. The study area for this project is approximately 30 acres. Existing improvements include paved access from Carlton Hills Boulevard, paved and unpaved parking lots, a restroom building, a basketball court, picnic areas, an off-leash fenced dog park, and a children's playground. The remainder of the park includes paved and unpaved walking trails and extensive open space with grass and isolated trees.

Based on our review of the Master Plan Report prepared by Schmidt Design Group (2012), the proposed Mast Park improvements include construction of a vehicular bridge to provide access for a new parking lot on the northwest area of the park, various pedestrian bridges, restroom building, picnic area, children's play area, relocated dog park, fitness stations and gazebos, enhancement of drainage channels, establishment of new stormwater retention basins and other minor improvements. The new parking lot may use a permeable pavement section.

The approximate site coordinates range between latitudes 32.844°N and 32.848°N, and between longitudes 116.991°W and 116.998°W. The park slopes gently to the south with surface grades ranging from elevation 343 feet above mean sea level (msl) at the north east corner to elevation 312 feet msl at the steeper riverbank area. Drainage across the site is by sheet flow in the southerly direction.

3. SCOPE OF SERVICES

Our scope of services for this project consisted of the following:

- We reviewed readily available background data including previous geotechnical reports for the site vicinity by others, as well as in-house geotechnical data, geologic maps, topographic maps, and aerial photographs relevant to the subject site.
- We performed a geotechnical site reconnaissance to observe the general surface conditions at the site and to select exploratory locations based on the plan provided by Dokken Engineering. After the planned locations were delineated, Underground Service Alert (USA) was notified a minimum of 48 hours prior to excavation.
- We obtained a boring permit from the San Diego County Department of Environmental Health (SDCDEH).
- We performed a subsurface evaluation including the excavation, logging, and sampling of six exploratory borings. We obtained samples of earth materials from the borings and transported them to our in-house laboratory for examination and testing.
- We excavated seven borings to perform percolation testing.

- We performed laboratory testing on selected samples of earth materials in order to evaluate the geotechnical engineering properties of the on-site soils.
- We compiled and analyzed the data collected from our site reconnaissance, subsurface evaluation, and laboratory testing. Specifically, our analyses included the following:
    - Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials;
    - Evaluation of geologic hazards and engineering seismology, including evaluation of fault rupture hazard, seismic shaking hazard, liquefaction and seismic settlement potential;
    - Evaluation of seismic design parameters in accordance with 2013 California Building Code;
    - Evaluation of current and historical groundwater conditions at the site and potential impact on design and construction;
    - Evaluation of expansion potential of on-site soils;
    - Evaluation of project feasibility and suitability of on-site soils for foundation support;
    - Development of general recommendations for earthwork, including requirements for placement of compacted fill;
    - Evaluation of foundation design parameters including allowable bearing capacity for shallow and deep foundations, estimated settlement, and lateral resistance;
    - Recommendations for temporary excavations;
    - Recommendations for concrete slab-on-grade support and concrete flatwork;
    - Recommendations for flexible and rigid pavement design; and,
    - Evaluation of the potential for the on-site materials to corrode buried concrete and metals.
- We prepared this report to present the work performed and data acquired and to summarize our conclusions and geotechnical recommendations for the design and construction of the proposed improvements.

4. FIELD EXPLORATION AND LABORATORY TESTING

4.1. Field Exploration

Our subsurface exploration was conducted on June 30 and July 8, 2016. The subsurface conditions were evaluated by advancing six 8-inch-diameter, hollow-stem auger borings to approximate depths ranging from 6½ to 51½ feet below existing ground surface (bgs) using a CME-75 truck-mounted drill rig. Driven samples of the soils were obtained using standard penetration test (SPT) and modified California split spoon samplers. The samplers were driven using a 140-pound, automatic-drop hammer falling approximately 30 inches. The blow counts were recorded and the materials encountered in the borings were logged by our field personnel. Upon completion of drilling, the borings were backfilled by the drilling subcontractor in accordance with SDCDEH requirements.

In addition, seven borings were excavated to depths ranging from 3 to 5 feet bgs to perform percolation testing. The approximate locations of the exploratory borings and percolation tests are shown on Figure 2, Exploration Location Map. The logs of borings are presented in Appendix A, Field Exploration.

4.2. Laboratory Testing

Laboratory tests were performed on selected samples obtained from the borings in order to aid in the soil classification and to evaluate the engineering properties of the foundation soils. Laboratory tests included in-situ moisture content and dry density, grain size analysis, expansion index, Atterberg limits, direct shear, consolidation, maximum density and optimum moisture content, R-value and soil corrosivity. In-situ moisture content and dry density data are presented on the boring logs in Appendix A. The remaining laboratory test results are presented in Appendix B.

4.3. Percolation Testing

Percolation testing was performed on June 30, 2016, in general conformance with the borehole percolation test procedure presented in Appendix D of the City of Santee BMP Design Manual (2016). The purpose of the testing was to evaluate the infiltration rates of subgrade soils for design of proposed stormwater infiltration systems. A detailed discussion of field testing and infiltration rates at the site is presented in Appendix C of this report.

5. REGIONAL GEOLOGIC SETTING AND SUBSURFACE CONDITIONS

The project area is located in the coastal plains portion of the Peninsular Ranges Geomorphic Province, within San Diego County. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin, south to the southern tip of Baja California (Norris and Webb, 1990). It varies in width from approximately 30 to 100 miles, and its primary structural fabric (faulting, jointing and ridge lines) trends northwest. The coastal plains portion of the province is bordered on the east by the western foothills and to the west by the Pacific Ocean.

The geology and subsurface formations of Santee include Eocene age sediments of the Friars Formation and Stadium Conglomerate, which are part of the marine terraces of the coastal plain landform. These sediments are generally underlain by Cretaceous granite, which comprises the basement rock of the Peninsular Ranges. The San Diego River carved out a river channel during the Pleistocene when sea water levels were fairly low; a rise in sea level at the end of the Pleistocene probably contributed to deposition of older alluvium. A subsequent rise in sea level is associated with deposition of the younger alluvium. The regional geology is shown in Figure 3, Regional Geologic Map.

5.1. Site Geology and Subsurface Condition

Earth materials encountered during the subsurface exploration consist of artificial fill, young alluvial deposits and Friars Formation materials. Generalized descriptions of these units are provided below. Detailed descriptions of the earth materials encountered in the exploratory borings are presented in Appendix A.

5.1.1. Artificial Fill

Artificial fill was encountered in exploratory boring B-1 from the surface to an approximate depth of 7 feet bgs. Artificial fill materials generally consist of light gray, sandy clay with gravel and trace cobbles and dark gray very dense silty sand.

5.1.2. Young Alluvium (Map Symbol Qya)

Young alluvial deposits underlie artificial fill in boring B-1 and were encountered at the surface in exploratory borings B-2 through B-6 and percolation test locations IF-1 through IF-7. Young alluvial deposit materials generally consist of light to dark gray and yellowish brown, very loose to very dense sand and silty sand, and stiff sandy clay.

5.1.3. Friars Formation (Map Symbol Tfr)

Friars Formation materials were observed to underlie the young alluvium at a depth of approximately 35 feet bgs in boring B-1, and at a depth of approximately 20 feet bgs in boring B-2. As observed in these excavations, the Friars Formation consists of light brown and light to dark gray, very stiff to hard, lean to fat clay with sand. These materials extended to the maximum exploration depth of 51½ feet bgs in both borings.

5.2. Groundwater

Groundwater was encountered in exploratory borings B-1 through B-5 and IF-5 at approximate depths ranging from 4½ to 10 feet bgs, corresponding to elevations 312 to 316½ feet (msl). Groundwater level data from a monitoring well located at the northwest corner of the park was obtained from the California Department of Water Resources Groundwater Information Center (2016) website and indicates that the groundwater level is at Elevation 316 feet (msl). Groundwater conditions may vary across the site due to stratigraphic and hydrologic conditions, and may change over time as a consequence of seasonal and meteorological fluctuations and activities by humans at this and nearby sites.

5.3. Rippability

Based on our subsurface exploration of the site, the young alluvial deposits and Friars Formation materials should be generally excavatable with heavy-duty earthwork equipment in good working condition.

5.4. Caving Potential

Due to the presence of cohesive materials onsite, caving during excavations is not anticipated. Shoring in accordance with CalOSHA guidelines is recommended for trench excavations. Drilling mud or casing may be needed to stabilize drilled holes for piers extending below the groundwater level.

5.5. Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. According to our observations and laboratory testing, the majority of the surface soils consist of granular materials having very low expansion potential, with the exception of clayey soils encountered in borings B-1 and IF-3. These soils may exhibit medium to high expansion potential. Where encountered under new structures, it is recommended that expansive soils be segregated and transported offsite for disposal.

6. ENGINEERING SEISMOLOGY AND DESIGN

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 1999). However, the site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered high during the design life of the proposed structure.

Geologic hazards at the site are essentially related to those caused by earthquakes. The major cause of damage from earthquakes is fault rupture and strong shaking from seismic waves. Significant faults in the vicinity of the site are shown in Figure 4, Fault Location Map. Based on our review of the City of Santee General Plan Geotechnical/Seismic Hazard Map (2003), the site is located within a zone of potential liquefaction as shown in Figure 5, Liquefaction Potential Map. Liquefaction hazard is discussed further in Sections 6.3 and 6.4 below.

6.1. Active Faulting

The southern California region has long been recognized as being seismically active. Seismic activity results from a number of active faults that cross the region, all of which are related to the San Andreas transform system which covers a broad zone of right lateral faults that extend from Cape Mendocino to Baja California. Faults in Southern California are classified according to their activity as active, potentially active, and inactive faults. Active faults are those faults that have had surface displacement within Holocene time (approximately the last 11,000 years) and/or have been included within an Alquist-Priolo Earthquake Fault Zone. Faults are considered potentially active if they show evidence of surface displacement since the beginning of Quaternary time (about 1.6 million years ago), but not since Holocene time. Faults are classified as inactive if they had surface movement prior to Quaternary time.

The site is not located within a currently established Alquist-Priolo Earthquake Fault Zone for fault rupture hazard (formerly Special Studies Zones for fault rupture hazard). Based on a review of geologic literature, no active or potentially active faults are known to occur beneath the project site. Accordingly, it appears that there is little probability of surface rupture due to faulting beneath the site. There are, however, several faults located in sufficiently close proximity that movement associated with them could cause significant ground motion at the site.

The closest known active faults to the site are the Rose Canyon fault zone, located 13 miles to the west, the Coronado Bank fault zone, located 25 miles to the southwest, the Elsinore fault zone, located 28 miles to the northeast and the Newport-Inglewood (Offshore) fault zone, located 33 miles to the northwest. Nearby faults that are considered to be potentially active include the La Nacion fault zone, which passes approximately 6 miles to the west. An inactive fault has been mapped trending towards Mast Park. The likelihood of fault rupture at the site is discussed below.

6.2. Surface Fault Rupture

Based on our field observations and review of published geologic literature, active faults do not pass through or near the site, nor do the surface traces of any known active or potentially active faults project towards the site. Therefore, the likelihood of fault rupture occurring at the site is considered to be low. The greatest seismic hazard likely to affect the site is seismic shaking which is accommodated for in the seismic design parameters presented in this report.

6.3. Liquefaction

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent, and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure and causes the soil to behave as a fluid for a short period of time. Based on the presence of granular alluvial deposits and the shallow groundwater level, the project site is considered potentially liquefiable. Evaluation of the seismic settlement due to liquefaction is presented in Section 6.4.

6.4. Seismic Settlement Potential

Seismic settlement can occur when loose to medium-dense granular materials densify during seismic shaking and liquefaction. Seismically-induced settlement may occur in dry, unsaturated, as well as saturated soils. Liquefaction is generally known to occur in loose, saturated, relatively clean, fine-grained cohesionless soils at depths shallower than approximately 50 feet. Factors to consider in the evaluation of soil liquefaction potential include groundwater conditions, soil type, grain size distribution, relative density, degree of saturation, and both the intensity and duration of ground motion. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity.

Liquefaction analyses were performed in accordance with the National Center for Earthquake Engineering Research (NCEER) procedure by Youd et al., (2001) using the computer program LiquefyPro (Civiltech, 2012), and the site data obtained from our borings and CPTs. The analyses considered an earthquake moment magnitude of 6.76, peak ground acceleration PGA_M of 0.39g, and groundwater level at the surface. Our analyses indicate that liquefaction may occur within granular soil layers extending to an approximate depth of 28 feet bgs. According to our analyses, the estimated settlement of liquefied soil layers will range from 2 to 3 inches during a seismic event as shown in Appendix D.

6.5. Landslides

Based on our review of the referenced geologic maps, literature, topographic maps, aerial photographs, and our subsurface evaluation, no landslides or related features underlie or are adjacent to the subject site. Due to the relatively level nature of the site, the potential for landslides at the project site is considered negligible.

6.6. Flooding and Seiches

The Federal Emergency Management Agency (FEMA) has prepared flood insurance rate maps (FIRMs) for use in managing the National Flood Insurance Program. An excerpt of the flood insurance rate map covering the project site is presented in Figure 6, FEMA FIRM Map. Based on our review of the FEMA flood map (United States Federal Emergency Management Agency, 2009b), the site is located within the 1 percent (100-year return period) annual chance floodplain. The southern portion of the site is located within the floodway of the San Diego River channel.

Seiches are standing wave oscillations of an enclosed water body after the original driving force has dissipated. The potential for the site to be adversely impacted by earthquake-induced seiches is considered to be negligible due to the lack of any significant enclosed bodies of water located in the vicinity of the site.

6.7. CBC Seismic Design Parameters

Our recommendations for seismic design parameters have been developed in accordance with 2013 California Building Code (CBC) and ASCE 7-10 (American Society of Civil Engineers, 2010) standards. As discussed above, the project site is potentially liquefiable and would be classified as Site Class F per ASCE 7-10 Section 20.3.1. However, ASCE 7-10 provides an exception for structures with fundamental periods of vibration equal to or less than 0.5 second. Since the proposed one-story structures are anticipated to have fundamental periods of less than 0.5 second, the site class may be determined using the definitions provided in Table 20.3.1 of ASCE 7-10. Based on the results of our field investigation, the applicable Site Class is D, consisting of a stiff soil profile with average SPT N between 15 and 50 blows per foot. Table 1 presents the seismic design parameters for the site in accordance with 2013 CBC and mapped spectral acceleration parameters (United States Geological Survey, 2016).

Table 1
2013 California Building Code Design Parameters

Design Parameters	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	0.874g
Mapped Spectral Acceleration Parameter at at Period 1-Second, S_1	0.340g
Site Coefficient, F_a	1.150
Site Coefficient, F_v	1.720
Adjusted MCE_R[1] Spectral Response Acceleration Parameter at Short Period, S_{MS}	1.006g
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	0.585g
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	2/3 S_{MS} = 0.670g
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	2/3 S_{M1} = 0.390g
Peak Ground Acceleration, PGA_M[2]	0.385g
Seismic Design Category	D

Notes: [1] Risk-Targeted Maximum Considered Earthquake
[2] Peak Ground Acceleration adjusted for site effects

7. DESIGN RECOMMENDATIONS

Based on the results of the field exploration and engineering analyses, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and are implemented during construction. It is our opinion that the proposed buildings may be supported on shallow spread footings or on mat foundations placed on engineered fill.

Our geotechnical engineering analyses performed for this report were based on the earth materials encountered during the subsurface exploration for the site. We have assumed maximum wall and column loads of 5 kips per foot and 20 kips, respectively, in our foundation analyses. If the design changes substantially, then our geotechnical engineering recommendations would be subject to revision based on our evaluation of the changes. The following sections present our conclusions and recommendations pertaining to the engineering design for this project.

7.1. Earthwork and Site Preparation

In general, earthwork should be performed in accordance with the recommendations presented in this report. Twining should be contacted with questions regarding the recommendations or guidelines presented herein.

7.1.1. Site Preparation

Site preparation should begin with the removal of utility lines, asphalt, concrete, vegetation, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside edges of the proposed excavation and fill areas. We recommend that unsuitable materials such as debris, organic matter or oversized material be selectively removed and disposed at a legal dump site away from the project area.

7.1.2. Removals and Overexcavation

The upper portion of the alluvial deposit material is considered compressible and not suitable for support of the building slab in its present condition. Therefore, for support of building slabs we recommend removal of loose alluvial soils to a depth of 3 feet (as measured from bottom of footing elevation) extending at least 5 feet outside the building envelope. The extent and depths of removal should be evaluated by Twining's representative in the field based on the materials exposed. Additional removals may be recommended if excessively loose or soft soils are exposed during grading.

7.1.3. Materials for Fill

On-site soils with "low" expansion potential (expansion index of 50 or less) and organic content of less than 3 percent by volume (or 1 percent by weight) are suitable for use as fill. Fill soil should not contain contaminated materials, rocks, lumps over 4 inches in largest dimension, or more than 40 percent larger than ¾ inch. Utility trench backfill material should not contain rocks or lumps over 3 inches in largest dimension. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed offsite. Any imported fill material should consist of "very low" expansion potential (expansion index of 20 or less) granular soil. Import material should also have low corrosion potential (chloride content less than 500 parts per million [ppm], soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher). Materials to be used as fill should be evaluated by a Twining representative prior to importing or filling. Cuttings generated from drilling operations will not be suitable as fill below structures, pavements or flatwork and should be exported offsite.

7.1.4. Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Twining. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve generally consistent moisture contents at or near the optimum moisture content. The scarified materials should then be compacted to 90 percent relative compaction in accordance with ASTM Test Method D1557. The evaluation of compaction by Twining should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify Twining and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

Fill materials should be moisture conditioned to near optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass. Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 6 to 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve near optimum moisture condition, mixed, and then compacted by mechanical methods using appropriate compacting rollers to a relative compaction of 90 percent as evaluated by ASTM D1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

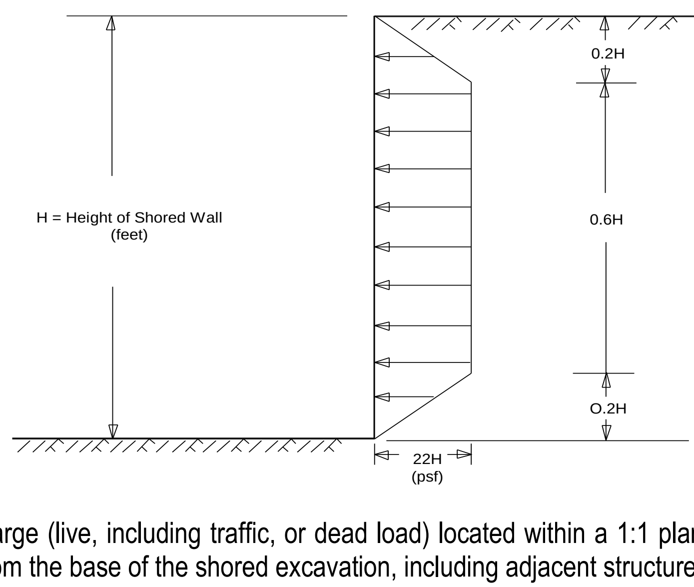
7.1.5. Excavations and Shoring

CalOSHA regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on a description of the soil types encountered. Trenches over 20 feet deep should be designed by the contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that OSHA Type C soil classification be used for excavations in alluvial deposits. Upon making the excavations, the soil classification and excavation performance should be evaluated in the field by Twining in accordance with OSHA regulations. For trench or other temporary excavations, OSHA requirements regarding personnel safety should be met by laying back the slopes to a gradient no steeper than 1.5:1 (horizontal:vertical) for fill and alluvial materials.

Where sloped excavations are created, the tops of the slopes should be barricaded so that vehicles and storage loads do not encroach within 10 feet of the tops of the excavated slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces. We recommend that excavated areas be backfilled as soon as practicable. The stability of the excavations decreases over time as the soil dries and weathers.

For vertical excavations less than approximately 15 feet in height, cantilevered shoring may be used. For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the drained soils, with a level surface behind the cantilevered shoring, will exert an equivalent fluid pressure of 35 pcf.

Tied-back or braced shoring should be designed to resist a trapezoidal distribution of lateral earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the following diagram with the maximum pressure equal to 22H in psf, where H is the height of the shored wall in feet.



Any surcharge (live, including traffic, or dead load) located within a 1:1 plane drawn upward and outward from the base of the shored excavation, including adjacent structures, should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind the temporary shoring may be calculated by multiplying the vertical surcharge pressure by 0.35. Lateral load contributions of surcharges located at a distance behind the shored wall may be provided once the load configurations and layouts are known. As a minimum, a 300 psf vertical uniform surcharge is recommended to account for nominal construction and/or traffic loads.

7.1.6. Excavation Bottom Stability

We anticipate that excavation bottoms could be unstable. Unstable conditions may be mitigated by overexcavation of the bottom by approximately 10 inches, placement of Tensar TX130 geogrid or similar material and replacement with a 10-inch layer of crushed aggregate base. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by Twining at the time of construction.

7.1.7. Construction Dewatering

Dewatering measures may be necessary during excavation operations. If needed, considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement of nearby structures, and groundwater discharge. Disposal of groundwater should be performed in accordance with guidelines of the Regional Water Quality Control Board.

7.2. Building Foundation Recommendations

The restroom building and other minor structures including may be supported on shallow foundation systems consisting a mat slab or spread footings connected by grade beams provided that the foundations are placed on compacted fill materials as described in Section 7.1. Shallow foundations should be designed using geotechnical design parameters presented in Table 2.

Table 2
Geotechnical Design Parameters for Continuous and Isolated Spread Footings

Minimum Footing Dimensions	• At least 12 inches in width and 18 inches in depth for continuous footings. • At least 24 inches in width and 24 inches in depth for square footings.
Allowable Bearing Pressure for Foundation Footings	• An allowable bearing pressure of 1,500 pounds per square foot (psf) can be used. • Allowable bearing values may be increased by one-third for transient live loads such as wind or seismic
Estimated Static Settlement (Total/Differential)	• Total static settlement is estimated to be less than 1 inch. • Differential settlement is estimated to be ½ inch in 40 feet between footings with similar loading conditions.
Estimated Dynamic Settlement (Total/Differential)	• Total dynamic settlement is estimated to be 2 to 3 inches. • Differential settlement is estimated to be ½ inch in 40 feet between footings with similar loading conditions.
Coefficient of Friction Below Footings	0.30
Unfactored Lateral Passive Resistance	200 pcf (equivalent fluid pressure)
Note: The total allowable lateral resistance can be taken as the sum of the friction resistance and passive resistance, provided that the passive resistance does not exceed two-thirds of the total allowable resistance.	

7.3. Vehicular Bridge Foundation

A vehicular bridge is proposed at the drainage channel crossing the access driveway to the new parking lot at the northwest area of the park. Due to the anticipated liquefaction settlement it is recommended that a deep foundation system consisting of driven piles be used for support of the proposed vehicular bridge. Deep foundations should extend through the alluvium and be founded in Friars Formation materials. Additional recommendations regarding the type of pile used for this project may be provided by the structural engineer.

7.3.1. Idealized Soil Profile

According to the results of our subsurface exploration we have developed the idealized soil profile presented in Table 3, which was used in the pile analyses.

Table 3
Idealized Soil Profile

Depth (feet)	Material Type	Unit Weight (pcf)	Cohesion, C (psf)	Friction Angle, ϕ (degrees)	Subgrade Modulus, k (pci)
0-7	Fill	132	1,000	0	300
7-9	Loose Sand Layer	132	0	25	20
9-35	Liquefiable Sand Layer	52.6*	0	25	20
35-45	Very Stiff Clay	70.4*	2,250	0	750
45-50	Hard Clay	71.1*	3,000	0	1,000
Note: * Buoyant unit weight below groundwater table assumed at 9 feet depth.					

7.3.2. Axial Pile Capacity

Pile capacity analyses were performed using the computer program AllPile (CivilTech, 2012). Axial pile capacity analyses were performed for driven 14-inch square pre-cast concrete piles (PCCPs) according to the following considerations:

- Compression capacity includes the downdrag force due to liquefaction during the design seismic event;
- Uplift capacity is based on the downward frictional capacity of the non-liquefiable layers and includes the pile weight;
- Driven piles should be embedded at least five feet into competent Friars Formation materials corresponding to minimum tip elevation 285 ft (msl);
- On-center pile spacing of three pile diameters or more should be maintained; and,
- The allowable downward load capacity curves presented in Appendix E were calculated for a factor of safety of 2.0 and the liquefied condition. Axial pile capacity may be increased by 1/3 when subject to short term loading such as seismic and wind forces.

7.3.3. Additional Considerations

Due to the variability of on-site soils, it is possible that pile capacity will be achieved prior to reaching specified tip elevation which may require additional driving effort or cutting of piles. Alternatively, the design capacity may not be achieved at the specified tip elevation and pile splicing may be necessary. The geotechnical engineer should observe pile driving to evaluate if adequate capacity has been attained. If unexpected soil and driving conditions are encountered, foundation modifications may be required.

A pile hammer that develops a minimum energy of 40,000 foot-pounds per blow is recommended. Pre-drilling or jetting should not be used during pile installation. Lateral pile capacity should be evaluated after design loads and pile selection are provided by the project structural engineer.

A dynamic pile load testing program to evaluate installed pile capacity is recommended. At least 2 piles should be tested and the tests should be performed in accordance with Caltrans criteria and/or ASTM procedures, as appropriate. Both end of driving (EOD) and beginning of re-strike (BOR) data should be collected to assess how much soil setup or relaxation occurred after initial driving.

7.4. Pipe Culvert Alternative

A cost-efficient alternative to the pile-supported vehicular bridge would be to construct the access driveway on an embankment with a pipe culvert for the drainage channel. This alternative is susceptible to damage due to liquefaction settlement however repair costs are anticipated to be manageable.

7.5. Light Pole Foundations

Light poles may be supported on 18-inch diameter or larger drilled piers extending at least four feet below finish grade. Axial capacity can be calculated using allowable shaft friction of 200 psf. A lateral bearing pressure of 200 psf/ft may be used for design.

7.6. Concrete Slabs

Conventional concrete slabs may be supported at grade on engineered fill in accordance with the recommendations of this report. For design of concrete slabs placed on compacted, engineered fill, a modulus of subgrade reaction (k) of 150 pounds per cubic inch (pci) may be used.

Floor slabs should be designed and reinforced in accordance with the structural engineer's recommendations. However, for slabs not supporting heavy loads, we recommend that the concrete should have a thickness of at least 4 inches, 28-day compressive strength of at least 3,000 pounds per square inch (psi), water-cement ratio of 0.50 or less, and slump of 4 inches or less. For slabs supporting equipment, a minimum thickness of 5 inches is recommended. Slabs should be minimally reinforced with No. 3 bars placed longitudinally at 18 inches on center. Control joints should be constructed in accordance with recommendations from the structural engineer or architect. Additional thickness and reinforcement recommendations may be provided by the structural engineer.

The slab subgrade should be tested for moisture and compaction immediately prior to placement of the gravel or sand base, if any. All underslab materials should be adequately compacted prior to the placement of concrete. Care should be taken during placement of the concrete to prevent displacement of underslab materials. The underslab material should be dry or damp and should not be saturated prior to the placement of concrete. The concrete slab should be allowed to cure properly and should be tested for moisture transmission prior to placing of moisture-sensitive floor coverings.

The recommendations presented above are intended to reduce the potential for cracking of slabs; however, even with the incorporation of the recommendations presented herein, slabs may still exhibit some cracking. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics.

Table 4 provides recommendations for various levels of protection against vapor transmission through concrete floor slabs placed over a properly prepared subgrade. Care should be taken not to puncture the plastic membrane during placement of the membrane itself and the overlying silty sand.

Table 4
Options for Subgrade Preparation below Concrete Floor Slabs

Primary Objective	Recommendation
Enhanced protection against vapor transmission	• Concrete floor slab-on-grade may be placed directly on a 15-mil-thick moisture vapor retarder that meets the requirements of ASTM E 1745 Class C (Stego Wrap or similar). • The moisture vapor retarder membrane may be placed directly on the subgrade (ACI302.1R-67); if required for either leveling of the subgrade or for protection of the membrane from protruding gravel, then place about 2 inches of silty sand[1] under the membrane. • Special consideration for curing the concrete, such as wet curing, should be made if concrete is placed directly on the impermeable vapor retarder.
Above-standard protection against vapor transmission	This option is available if the slab perimeter is bordered by continuous footings at least 24 inches deep, OR if the area adjacent and extending at least 10 feet from the slab is covered by hardscape without planters: • 2 inches of dry silty sand[1]; over • Waterproofing plastic membrane 10-mil thick; over • At least 4 inches of ¾-inch crushed rock[2] or clean gravel[3] to act as a capillary break
Standard protection against vapor transmission	• 2 inches of dry silty sand[1]; over • Waterproofing plastic membrane 10-mil thick. If required for either leveling of the subgrade or for protection of the membrane from protruding gravel, place at least 2 inches of silty sand[1] under the membrane.
Coefficient of Friction Below Footings	0.30
Unfactored Lateral Passive Resistance	200 pcf (equivalent fluid pressure)

Notes:

1 Silty sand should have a gradation between approximately 15 and 40 percent passing the No. 200 sieve and a plasticity index (PI) of less than 4.

2 ¾-inch crushed rock should conform to Section 200-1.2 of the latest edition of the "Green Book" Standard Specifications for Public Works Construction (Public Works Standards, Inc., 2015).

3 Gravel should contain less than 10 percent of material passing the No. 4 sieve and less than 3 percent passing the No. 200 sieve.

7.7. Retaining Walls

7.7.1. Lateral Earth Pressure

For retaining walls less than 6 feet in height, the following recommendations can be used for structural design. The values presented below assume that the supported grade is level and that surcharge loads are not applied. The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind the retaining walls and that external hydrostatic pressure will not develop behind the walls.

Walls that are supporting earth that has adequate drainage, and are restrained against rotation at the top (such as by a floor deck), may be designed for “at-rest” lateral earth pressure equivalent to a fluid weighing 55 pcf. For walls that are free to rotate at the top (such as cantilevered walls), the lateral earth pressure may be designed for the “active” equivalent fluid pressure of 35 pcf. Where adequate drainage is not provided behind walls, further evaluation should be conducted by the geotechnical engineer.

Vertical surcharge loads within a 1:1 (horizontal:vertical) projection upward from the bottom of the wall distributed over retained soils should be considered as additional uniform horizontal pressure acting on the wall. The additional horizontal pressure acting on the wall can be estimated as approximately 35% and 50% of the magnitude of the vertical surcharge pressure for the “active” and “at-rest” conditions, respectively. All permanent surcharge loading conditions should be evaluated on a case-by-case basis by the geotechnical engineer.

7.7.2. Seismic Lateral Earth Pressure

We do not anticipate retaining walls greater than 6 feet in height. If incorporated to the project; retaining walls greater than 6 feet in height should be designed to support seismic lateral earth pressures in accordance with 2013 CBC requirements.

7.7.3. Backfill and Drainage of Walls

Backfill material behind walls should consist of granular “very low” expansion potential material (Expansion Index no greater than 20) and should be approved by the project geotechnical engineer. Retaining walls should be waterproofed and adequately drained in order to limit hydrostatic buildup behind walls. Wall drainage may be provided by a geosynthetic drainage composite such as TerraDrain®, MiraDrain®, or equivalent, attached to the outside perimeter of the wall. The drain should be placed continuously along the back of the wall and connected to a 4-inch-diameter perforated pipe. The pipe should be sloped at least 1% and should be surrounded by 1 cubic foot per foot of ¾-inch crushed rock wrapped in suitable non-woven filter fabric (Mirafi® 140NL or equivalent). Crushed rock should meet the requirements defined in Section 200-1.2 of the latest edition of The “Greenbook” Standard Specifications for Public Works Construction (Public Works Standards, 2012). The drain should discharge through a solid pipe to an appropriate outlet.

7.8. Concrete Flatwork

Exterior concrete flatwork should be 5 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 18 inches on center both ways. To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be constructed with dowels and crack control joints at 10-foot spacing or as designed by the structural engineer along with keeping pad grade soils at elevated moisture content. Positive drainage should be established and maintained adjacent to flatwork. Uniform moisture content should be maintained throughout the year to reduce differential heave of concrete flatwork.

7.9. Preliminary Pavement Design

Based on the results of R-value testing on a sample of subgrade materials from the site, we have used an R-value of 12 for design of flexible pavements as shown in Table 5. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations in the areas to be paved once grading operations have been performed. We considered Traffic Index (TI) of 5 for parking stalls and light traffic driveways and TI of 6 for truck traffic and access driveways. Traffic indices used for this project should be determined by the project civil engineer based upon anticipated traffic loading conditions. Additional pavement section recommendations for different traffic indices can be provided by Twining, if requested.

Table 5
Pavement Recommendations

Traffic Index	Pavement Area	Design R-value	Asphalt Concrete (inches)	Aggregate Base (inches)
5.0	Light Traffic	12	3.0	9.0
6.0	Truck Traffic	12	4.0	10.0

The aggregate base and upper 12 inches of the subgrade materials should be compacted to a relative compaction of 95 percent as evaluated by ASTM D1557. We suggest that consideration be given to using Portland cement concrete (PCC) pavements in areas where dumpsters will be stored and where buses and garbage trucks will stop and load. We recommend for these areas a 6½ -inch thick PCC pavement section with flexural strength of 600 psi placed over 6 inches of aggregate base compacted to 95 percent relative compaction as evaluated by ASTM D1557.

7.10. Drainage Control

The control of surface water is essential to the satisfactory performance of the building and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the improvements, even during periods of heavy rainfall. The following recommendations are considered minimal:

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.

- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located adjacent to the structures wherever possible. If planters are to be located adjacent to the structures, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.
- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

7.11. Corrosion

Laboratory testing was performed on a representative samples of on-site soils to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test 643 and the sulfate and chloride tests were performed in accordance with California Tests 417 and 422, respectively. These laboratory test results are presented in Appendix B.

The results of the corrosivity testing indicated electrical resistivity value of 2,750 ohm-cm. The soil pH was 6.4. The tests indicated soluble chloride content of 369 parts per million (ppm) and soluble sulfate content of 74 ppm (that is, 0.007 percent). Based on Caltrans (2012) criteria, the on-site soils would not be classified as corrosive, which is defined as soil having more than 500 ppm chlorides, more than 0.2 percent sulfates, or pH less than 5.5.

7.12. Concrete

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical deterioration. Laboratory testing indicated a sulfate content of the samples tested of 0.73 and 1.28 percent, which corresponds to sulfate exposure Class S0 – Negligible (sulfate content below 0.1%). Although the results were not significantly high, due to the variability in the on-site soils and the potential future use of reclaimed water at the site, we recommend that 3 inches of concrete cover be provided over reinforcing steel, and that Type II/V cement be used for cast-in-place structures in contact with soil. In addition, we recommend a water to cement ratio of no more than 0.50. A corrosion specialist may be consulted regarding suitable types of piping and appropriate protection for underground metal conduits.

8. DESIGN REVIEW

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation of excavations will be important to the performance of the proposed development. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

8.1. Plans and Specifications

The design plans and specifications should be reviewed by Twining prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in the light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications. Based on the work already performed, this office is best qualified to provide such review.

8.2. Construction Monitoring

Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, foundation installation, and other site grading operations should be observed and tested. The substrata exposed during the construction may differ from that encountered in the test excavations. Continuous observation by a representative of Twining during construction allows for evaluation of the soil conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

The project geologist should be notified prior to exposure of subgrade. It is critically important that the geologist be provided with an opportunity to observe and/or map all exposed subgrade prior to burial or covering.

9. LIMITATIONS

The recommendations and opinions expressed in this report are based on information obtained from our field exploration for the entire site. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during excavation operations, for example, the presence of unsuitable soil, and that additional effort may be required to mitigate them.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Twining has no control.

Twining's recommendations for this site are, to a high degree, dependent upon appropriate quality control of foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Twining to observe foundation excavations for the proposed construction. If parties other than Twining are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Twining should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report has been prepared for the exclusive use by the client and its agents for specific application to the proposed design and construction of the project described herein. Any party other than the client who wishes to use this report for an adjacent or nearby project, shall notify Twining of such intended use. Land use, site conditions, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of this report and the nature of the project, Twining may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or any other party will release Twining from any liability resulting from the use of this report by any unauthorized party.

Twining has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

10. SELECTED REFERENCES

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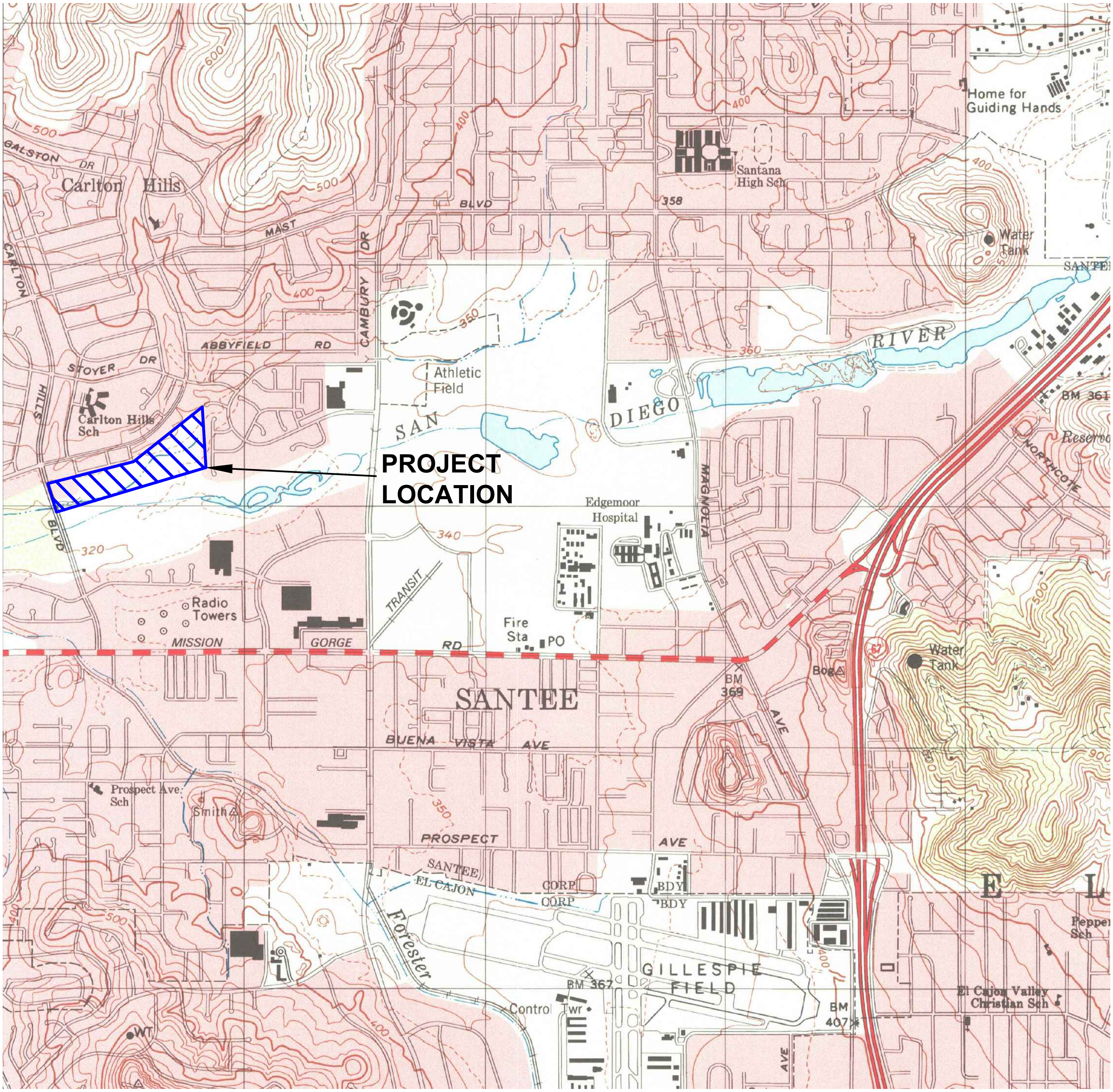
FIGURES



APPROXIMATE PROJECT COORDINATES
LATITUDE: 32.845°N
LONGITUDE: 116.994°W

N

SCALE IN FEET
0 2,000 4,000

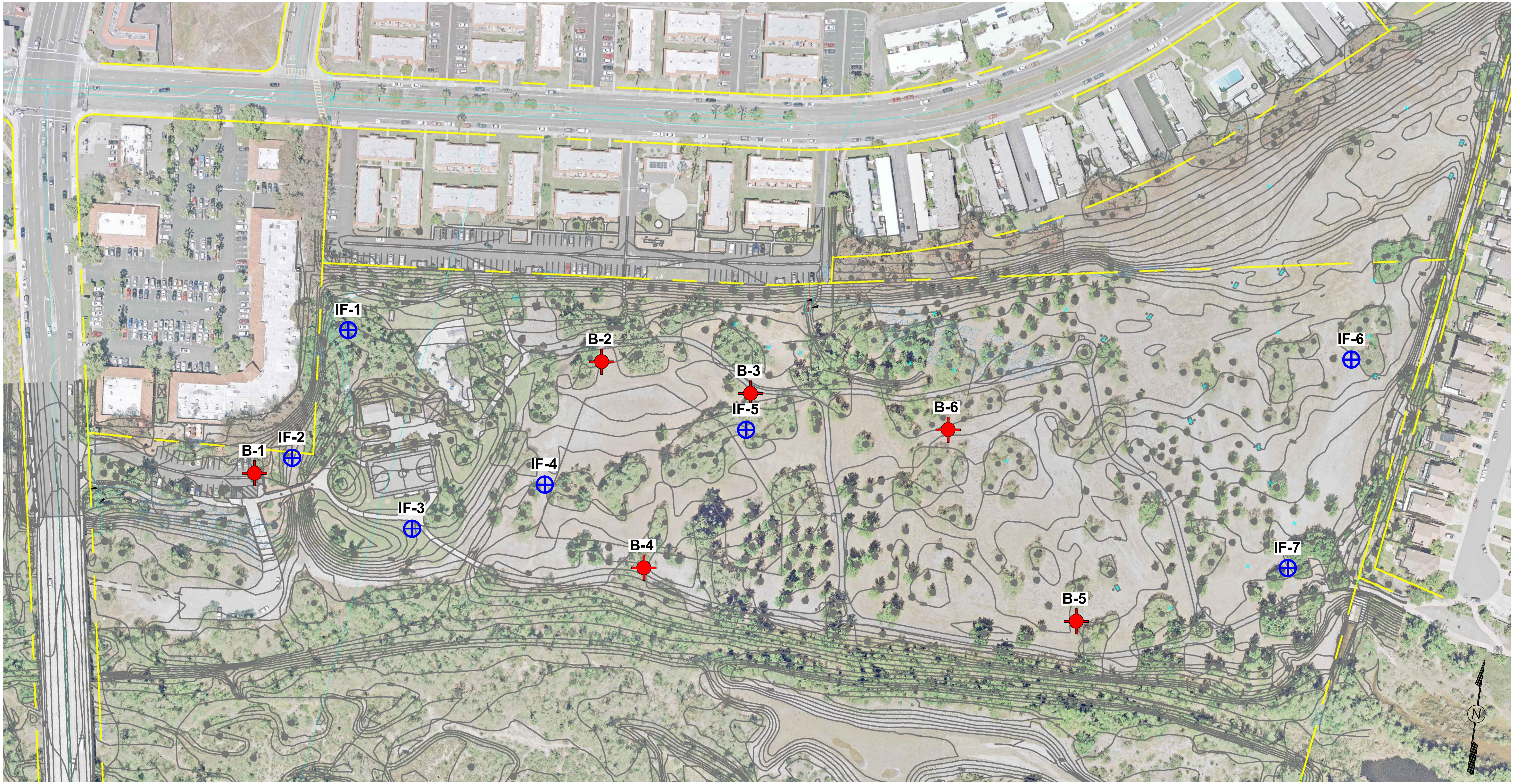
REFERENCE: UNITED STATES GEOLOGICAL SURVEY (1996)

TWINING

PROJECT LOCATION MAP

MAST PARK IMPROVEMENTS
SANTEE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 1
160392.2	August 2016	



NOTE: All dimensions, locations, and directions are approximate.

LEGEND

B-1 BORING LOCATION

IF-1 INFILTRATION TEST LOCATION

TWINING

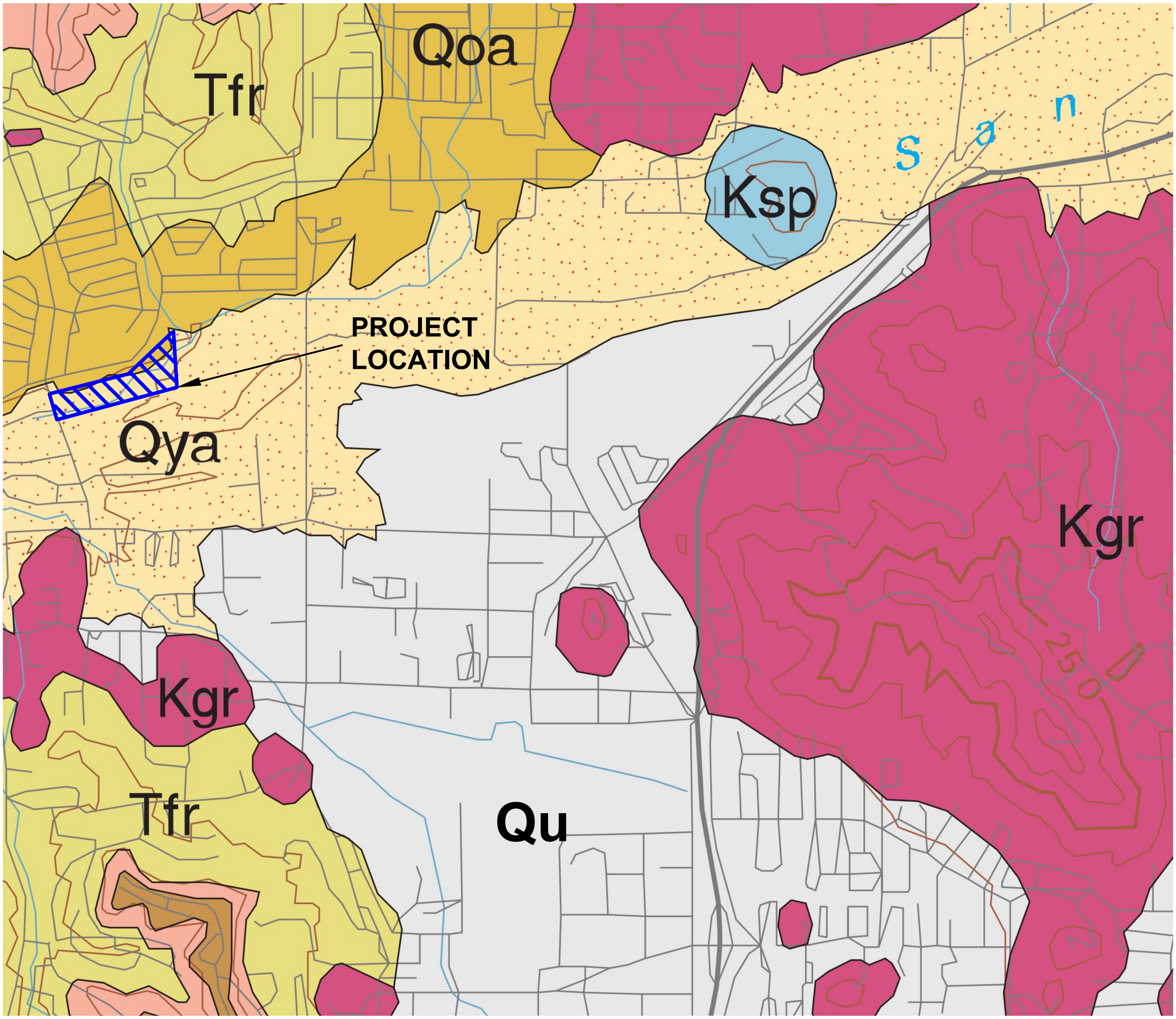
SCALE IN FEET
0 150 300

Reference: Schmidt Design Group, Inc. (2012)

EXPLORATION LOCATION MAP

MAST PARK IMPROVEMENTS
SANTEE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 2
160392.2	August 2016	



LEGEND

Qya	YOUNG ALLUVIUM	Ksp	SANTIAGO PEAK VOLCANICS
Qoa	OLDER ALLUVIUM	Tfr	FRIARS FORMATION
Qu	ALLUVIUM AND COLLUVIUM, UNDIVIDED		CONTACT-CONTACT BETWEEN GEOLOGIC UNITS; DOTTED WHERE CONCEALED.
Kgr	GRANITOID ROCKS		

N

SCALE IN MILES

0 1/2 1

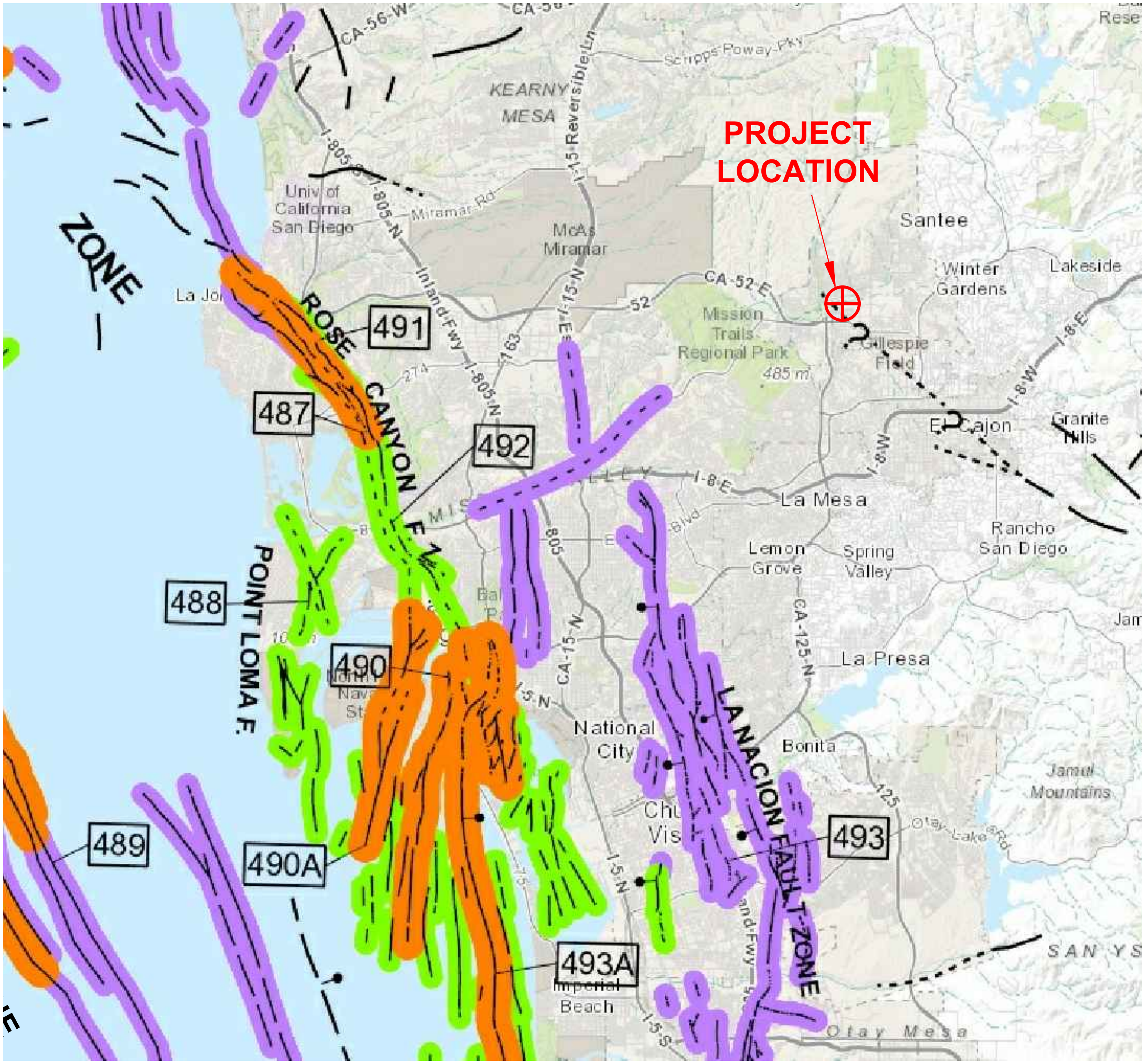
REFERENCE: CALIFORNIA GEOLOGICAL SURVEY (2004)

TWINING

REGIONAL GEOLOGIC MAP

MAST PARK IMPROVEMENTS
SANTEE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 3
160392.2	August 2016	



LEGEND

FAULT ALONG WHICH HISTORIC DISPLACEMENT HAS OCCURRED

HOLOCENE FAULT DISPLACEMENT

LATE QUATERNARY FAULT DISPLACEMENT

QUATERNARY FAULT DISPLACEMENT

PRE-QUATERNARY FAULT DISPLACEMENT

N

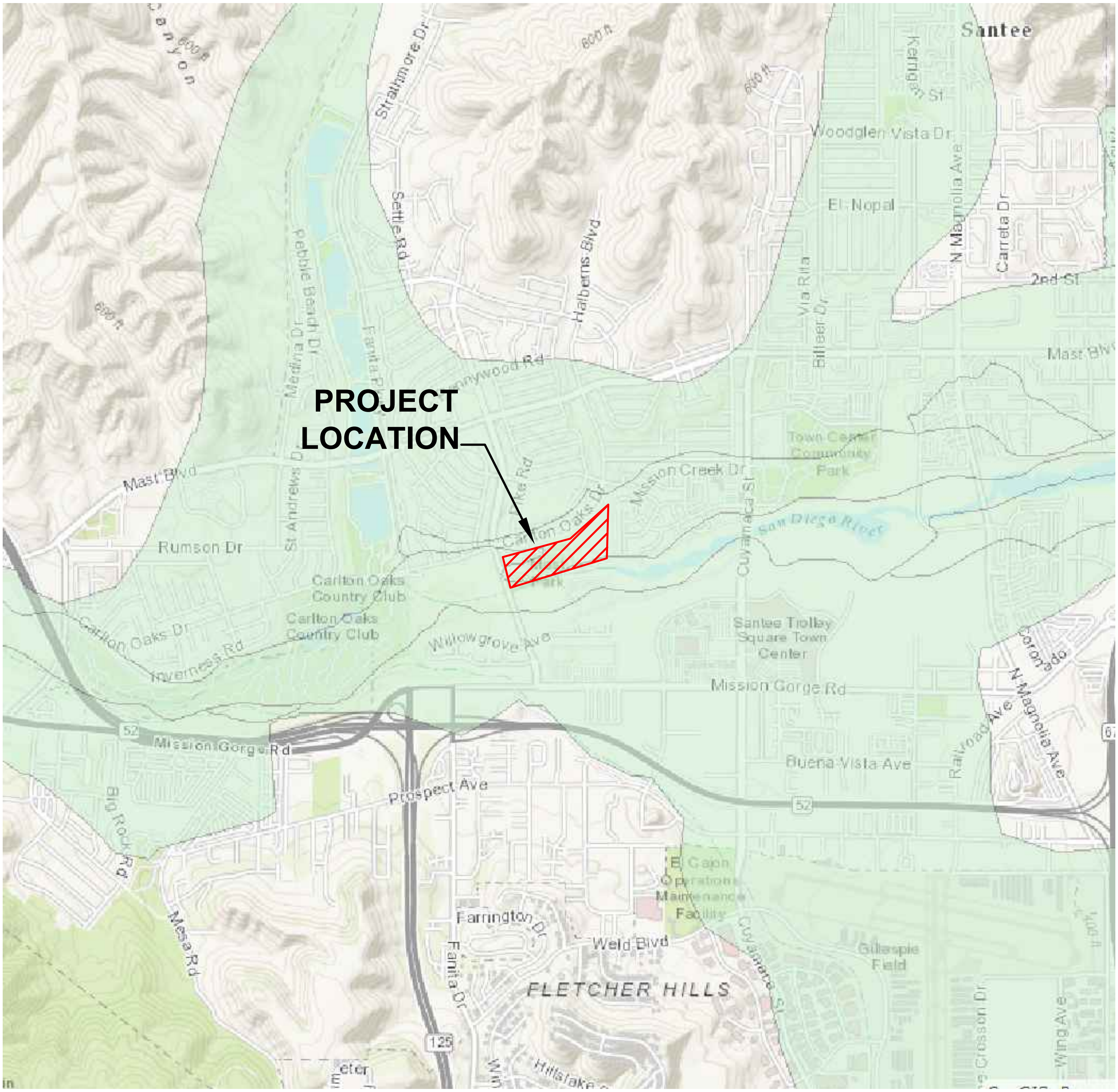
SCALE IN MILES

0 4 8

REFERENCE: CALIFORNIA GEOLOGICAL SURVEY (2010)

TWINING

FAULT LOCATION MAP		
MAST PARK IMPROVEMENTS SANTEE, CALIFORNIA		
PROJECT NO. 160392.2	REPORT DATE August 2016	FIGURE 4



LEGEND

ZONE OF POTENTIAL LIQUEFACTION

N

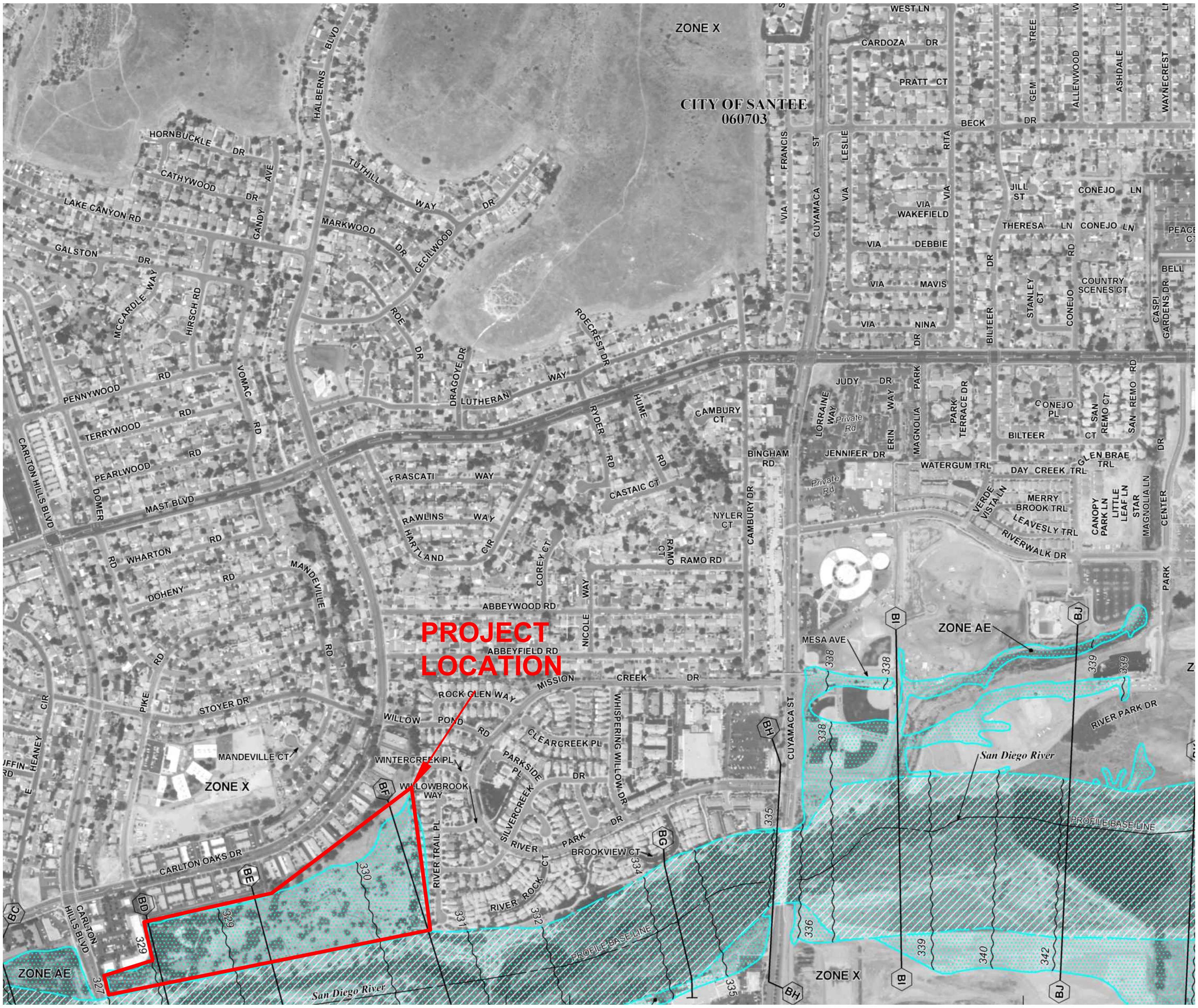
SCALE IN MILES

0 1/2 1

REFERENCE: ARCGIS, SAN DIEGO LIQUEFACTION (2014)

TWINING

LIQUEFACTION POTENTIAL MAP		
MAST PARK IMPROVEMENTS SANTEE, CALIFORNIA		
PROJECT NO. 160392.2	REPORT DATE August 2016	FIGURE 5



ZONE AE — AREAS SUBJECT TO FLOODING BY THE 1% ANNUAL CHANCE FLOOD

ZONE AE — FLOODWAY AREA IN ZONE AE

ZONE X — AREAS SUBJECT TO FLOODING BY THE 0.2% ANNUAL CHANCE FLOOD

ZONE X — AREAS OUTSIDE OF THE 0.2% ANNUAL CHANCE FLOODPLAIN

N

SCALE IN FEET

0 1,000 2,000

REFERENCE: FEMA, FIRM FLOOD INSURANCE RATE MAP, PANEL 1651G, SAN DIEGO COUNTY, CA (2012).

TWINING

FEMA FIRM Map

MAST PARK IMPROVEMENTS
SANTEE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 6
160392.2	August 2016	

APPENDIX A
FIELD EXPLORATION

Appendix A
Field Exploration

General

The subsurface exploration program for the proposed project consisted of drilling and logging six 8-inch diameter exploratory borings and seven borehole percolation tests. The 8-inch diameter exploratory borings were advanced using a truck-mounted CME-75 drill rig equipped with hollow stem augers. Drilling was performed by Baja Exploration of Escondido, California. The borings reached depths of approximately 3 to 51½ feet below existing grades. Upon completion of the borings, the boreholes were backfilled in accordance with SDCDEH requirements.

Drilling and Sampling

The boring logs are presented as Figures A-2 through A-15. An explanation of these logs is presented as Figure A-1. The boring logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The log also shows the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by a Twining engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

A California modified sampler was used to obtain drive samples of the soil encountered. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft that is driven a total of 18-inches into the soil at the bottom of the boring. The soil was retained in brass rings for laboratory testing. Additional soil from each drive remaining in the cutting shoe was usually discarded after visually classifying the soil. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs.

Disturbed samples were obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is advanced into the soil at the bottom of the drilled hole a total of 18 inches. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs. Soil samples obtained by the SPT were retained in plastic bags.

Both the California modified and the SPT sampler were driven by an automatic-trip hammer weighing 140 pounds at a drop height of approximately 30 inches.

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UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

COARSE-GRAINED SOILS | **FINE-GRAINED SOILS**

Relative Density	SPT (blows/ft)	Relative Density (%)	Consistency	SPT (blows/ft)
Very Loose	<4	0 - 15	Very Soft	<2
Loose	4 - 10	15 - 35	Soft	2 - 4
Medium Dense	10 - 30	35 - 65	Medium Stiff	4 - 8
Dense	30 - 50	65 - 85	Stiff	8 - 15
Very Dense	>50	85 - 100	Very Stiff	15 - 30
			Hard	>30

NOTE: SPT blow counts based on 140 lb. hammer falling 30 inches

Sample Symbol	Sample Type	Description
	SPT	1.4 in I.D., 2.0 in. O.D. driven sampler
	California Modified	2.4 in. I.D., 3.0 in. O.D. driven sampler
	Bulk	Retrieved from soil cuttings
	Thin-Walled Tube	Pitcher or Shelby Tube

LABORATORY TESTING ABBREVIATIONS

ATT	Atterberg Limits
C	Consolidation
CORR	Corrosivity Series
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Distribution
K	Permeability
MAX	Moisture/Density (Modified Proctor)
O	Organic Content
RV	Resistance Value
SE	Sand Equivalent
SG	Specific Gravity
TX	Triaxial Compression
UC	Unconfined Compression

TWINING

EXPLANATION FOR LOG OF BORINGS

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A-1

DATE DRILLED	7/8/16	LOGGED BY	SM	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	9.8
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	325 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									ASPHALT CONCRETE: 2.5 inches
									BASE: 4 inches
								CL	ARTIFICIAL FILL: Sandy lean CLAY with gravel, light gray, moist, trace cobbles
320	5	Driven	50/5"	13.5	116.3	GS RV		SM	Silty SAND, dark gray, moist, very dense, trace gravel
		Bulk						SM	YOUNG ALLUVIUM: Silty SAND, light gray, moist, medium dense
315	10	Driven	18			GS		SP-SM	Poorly graded SAND with silt, yellowish-brown, saturated, medium dense, medium- to coarse-grained
310	15	Driven	34	16.9	109.5	CONSOL			-- dark grey, medium-grained
305	20	Driven	9			GS		SP	Poorly graded SAND, dark gray, saturated, loose, fine- to medium-grained
300	25	Driven	27	15.9	111.9	DS			-- medium dense
295	30	Driven	29			GS			
290	35								

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 2

DATE DRILLED	7/8/16	LOGGED BY	SM	BORING NO.	B-1
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	9.8
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	325 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
			X	35	20.6	102.7	ATT, CONSOL		CH	FRIARS FORMATION: Fat CLAY with sand, gray, moist, very stiff, medium-grained sand
285	40			30						-- hard
280	45		X	33	18.2	108.9	DS		CL	Lean CLAY, light gray, moist, very stiff
275	50			37			ATT			-- trace sand, hard
										Total Depth = 51.5 feet Backfilled on 7/8/2016 Groundwater observed at depth of 9.8' at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
270	55									
265	60									
260	65									
255	70									

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 2

DATE DRILLED	7/8/16	LOGGED BY	SM	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	6.5
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	322 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0	X					CORR		SM	YOUNG ALLUVIUM: Silty SAND, reddish-brown, moist, very loose, fine-grained
317	5		X	3			GS			-- medium brown, wet, clay lens in upper 3 inches
312	10		X	20	12.1	118.8	CONSOL		SP	Poorly graded SAND, yellowish-gray mottled black, saturated, medium dense
307	15		X	28						-- yellowish-gray
										-- increase in clay content
302	20		X	40	23.9	94.2			CH	FRIARS FORMATION: Fat CLAY, light brown mottled black, moist, very stiff, iron-oxide staining
297	25		X	24			ATT			-- dark gray mottled pink
292	30		X	25	21.4	104.6				-- dark gray mottled light gray
287	35									

DATE DRILLED	7/8/16	LOGGED BY	SM	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	6.5
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	322 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				18					CH	FRIARS FORMATION: Fat CLAY, light brown mottled black, moist, very stiff, iron-oxide staining *(continued)* -- dark gray
282	40			32	26.8	95.0				-- lens of light yellow siltstone
277	45			19					CL	Lean CLAY, light gray, damp, very stiff
272	50			36	20.7	100.1				
										Total Depth = 51.5 feet Backfilled on 7/8/2016 Groundwater observed at depth of 6.5' at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
267	55									
262	60									
257	65									
252	70									

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE A - 3
160392.2	August 2016	

DATE DRILLED	7/8/16	LOGGED BY	SM	BORING NO.	B-3
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	5.7
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	321 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0	X					MAX		SM	YOUNG ALLUVIUM: Silty SAND, reddish-brown, damp, fine- to medium-grained
316	5		X	8	16.0	96.1				-- saturated, loose
									SP	Poorly graded SAND, yellow-brown, saturated, loose, coarse-grained
311	10		X	11						-- dark gray, medium dense
306	15		X	21						
										Total Depth = 16.5 feet Backfilled on 7/8/2016 Groundwater observed at depth of 5.7' at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
301	20									
296	25									
291	30									
286	35									

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE A - 4
160392.2	August 2016	

DATE DRILLED	7/8/16	LOGGED BY	SM	BORING NO.	B-4
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	9.8
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	322 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0–5	Bulk						SM	YOUNG ALLUVIUM: Silty SAND, medium brown, damp, fine- to medium-grained
317	5		Driven	9				SP	Poorly graded SAND, medium brown, moist, loose, coarse-grained, trace silt
312	10		Driven	12	20.0	99.5			-- dark gray, saturated, fine- to medium-grained
307	15		Driven	18					-- saturated, medium dense, coarse-grained
									Total Depth = 16.5 feet Backfilled on 7/8/2016 Groundwater observed at depth of 9.8' at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
302	20								
297	25								
292	30								
287	35								

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 5

DATE DRILLED 7/8/16 | LOGGED BY SM | **BORING NO. B-5**

DRIVE WEIGHT 140 lbs. | DROP 30 inches | DEPTH TO GROUNDWATER (ft.) 10.1

DRILLING METHOD 8" HSA | DRILLER Baja Drilling | SURFACE ELEVATION (ft.) 324 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0	X						SP	YOUNG ALLUVIUM: Poorly graded SAND, dark brown, moist, fine- to medium-grained
319	5		X	14	2.6	106.1		SP-SM	Poorly graded SAND with silt, dark brown, moist, loose, fine-grained
314	10		X	3				SP	Poorly graded SAND, dark brown, saturated, very loose, coarse-grained, trace silt
309	15		X	19	14.0	107.4			-- medium dense, dark gray
									Total Depth = 16.5 feet Backfilled on 7/8/2016 Groundwater observed at depth of 10.1' at Borehole backfilled in accordance with SDCDEH requirements.
304	20								
299	25								
294	30								
289	35								

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 6

DATE DRILLED	7/8/16	LOGGED BY	SM	BORING NO.	B-6
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	321 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SP-SM	YOUNG ALLUVIUM: Poorly graded SAND with silt, dark brown, moist, loose
				8					
316	5			28	4.9	94.6			-- medium dense
									Total Depth = 6.5 feet Backfilled on 7/8/2016 Borehole backfilled in accordance with SDCDEH requirements.
311	10								
306	15								
301	20								
296	25								
291	30								
286	35								

TWINING

LOG OF BORING

Mast Park Improvement Project
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PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 7

DATE DRILLED	6/30/16	LOGGED BY	AM	**BORING NO.**	**IF-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	323 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SM	YOUNG ALLUVIUM: Silty SAND, dark brown, damp, fine-grained
									Total Depth = 3.0 feet Backfilled on 7/1/2016 Borehole backfilled in accordance with SDCDEH requirements.
318	5								
313	10								

BORING LOG MAST PARK BORING LOGS.GPJ TWINING LABS.GDT 8/10/16

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 8

DATE DRILLED	6/30/16	LOGGED BY	AM	**BORING NO.**	**IF-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	321 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								ML	YOUNG ALLUVIUM: SILT with gravel and cobble, medium brown, damp, very dense
								SM	Silty SAND, medium brown, moist, fine-grained
									Total Depth = 3.0 feet Backfilled on 6/30/2016 Borehole backfilled in accordance with SDCDEH requirements.
316	5								
311	10								

BORING LOG MAST PARK BORING LOGS.GPJ TWINING LABS.GDT 8/10/16

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 9

DATE DRILLED	6/30/16	LOGGED BY	AM	BORING NO.	IF-3
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	324 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SM	YOUNG ALLUVIUM: Silty SAND, medium brown, moist, few clay
									Total Depth = 3.0 feet Backfilled on 7/1/2016 Borehole backfilled in accordance with SDCDEH requirements.
319	5								
314	10								

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 10

DATE DRILLED	6/30/16	LOGGED BY	AM	**BORING NO.**	**IF-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	319 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SM	YOUNG ALLUVIUM: Silty SAND, medium brown, moist, fine-grained
									Total Depth = 3.0 feet Backfilled on 6/30/2016 Borehole backfilled in accordance with SDCDEH requirements.
314	5								
309	10								

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 11

DATE DRILLED	6/30/16	LOGGED BY	AM	**BORING NO.**	**IF-5**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	4.5
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	321 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SP-SM	YOUNG ALLUVIUM: Poorly graded SAND with silt, medium brown, moist, fine-grained
									-- wet
									-- saturated
316	5								Total Depth = 5.0 feet Backfilled on 6/30/2016 Groundwater observed at depth of 4.5' at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
311	10								

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 12

DATE DRILLED	6/30/16	LOGGED BY	AM	BORING NO.	IF-6A
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	323 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SP-SM	YOUNG ALLUVIUM: Poorly graded SAND with silt and gravel, medium brown, damp, fine-grained
									Total Depth = 3.0 feet Backfilled on 7/1/2016 Borehole backfilled in accordance with SDCDEH requirements.
318	5								
313	10								

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 14

DATE DRILLED	6/30/16	LOGGED BY	AM	**BORING NO.**	**IF-6B**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	7
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	323 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SP-SM	YOUNG ALLUVIUM: Poorly graded SAND with silt and gravel, medium brown, damp, fine-grained
									-- no gravel
318	5								
									-- wet
									▼
									-- saturated
									Total Depth = 8.0 feet Backfilled on 7/1/2016 Groundwater observed at depth of 7' at completion of drilling. Borehole backfilled in accordance with SDCDEH requirements.
313	10								

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 13

DATE DRILLED	6/30/16	LOGGED BY	AM	**BORING NO.**	**IF-7**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	327 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SP-SM	YOUNG ALLUVIUM: Poorly graded SAND, medium brown, damp, fine-grained
									Total Depth = 3.0 feet Backfilled on 7/1/2016 Borehole backfilled in accordance with SDCDEH requirements.
322	5								
317	10								

TWINING

LOG OF BORING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE A - 15

APPENDIX B
LABORATORY TESTING

Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry densities of selected driven samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D2937. The test results are presented on the logs of the exploratory borings in Appendix A and also summarized in Table B-1.

Table B-1
Laboratory Moisture Content and Dry Density

Boring No.	Depth (feet)	Moisture Content (%)	Dry Unit Weight (pcf)
B-1	5	13.5	116.3
B-1	15	16.9	109.5
B-1	25	15.9	111.9
B-1	35	20.6	102.7
B-1	45	18.2	108.9
B-2	10	12.1	118.8
B-2	20	23.9	94.2
B-2	30	21.4	104.6
B-2	40	26.8	95.0
B-2	50	20.7	100.1
B-3	5	16.0	96.1
B-4	10	20.0	99.5
B-5	5	2.6	106.1
B-5	15	14.0	107.4
B-6	5	4.9	94.6

Atterberg Limits

Atterberg limits tests were performed on selected soil samples to evaluate plasticity characteristics and to aid in the classification of the soil. The tests were performed in general accordance with ASTM D4318. The results are presented in Figure B-1.

Sieve Analyses and Hydrometer Tests

The grain-size distribution of selected soil samples was evaluated in general accordance with ASTM C136/C117 and ASTM D422. Test results are presented on Figures B-2 through B-13.

Expansion Index Test

The expansion index of selected soil samples was evaluated in general accordance with ASTM D4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch-thick by 4-inch-diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the Expansion Index test are presented on Table B-2.

Table B-2
Expansion Index Test Result

Boring No.	Depth (feet)	Expansion Index	Expansion Potential
B-2	0 - 5	3	Very Low

Direct Shear Tests

Direct shear tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D3080 to evaluate the shear strength characteristics of the material. The samples were inundated during shearing to represent adverse field conditions. Test results are presented on Figures B-14 and B-15.

Consolidation Tests

Consolidation tests were performed on selected samples in general accordance with the latest version of ASTM D2435. The sample was inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The test results are presented on Figures B-16 through B-18.

Corrosivity

Soil pH and resistivity tests were performed by Anaheim Test Laboratories on a representative soil sample in general accordance with the latest version of California Test Method 643. The chloride content of the selected samples was evaluated in general accordance with the latest version of California Test Method 422. The sulfate content of the selected samples was evaluated in general accordance with the latest version of California Test Method 417. The test results are presented on Table B-3.

Table B-3
Corrosivity Test Results

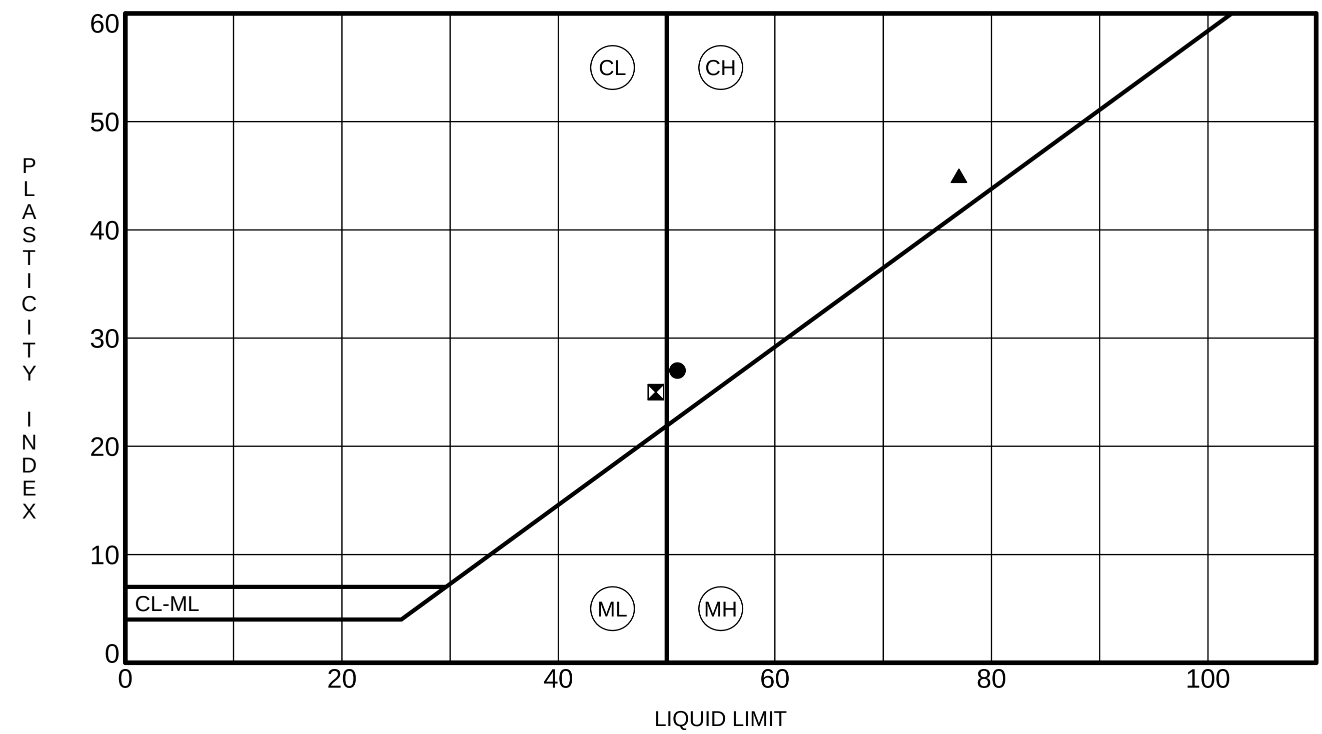
Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-2	0 - 5	6.4	74	369	2,750

Resistance Value (R-Value)

R-value testing was performed on a select bulk sample of the near-surface soils encountered at the site. The test was performed in general accordance with ASTM D2844. The results are summarized in Table B-4.

Table B-4
R-Value Test Results

Boring No.	Depth (feet)	R – Value
B-1	0 – 5	12



	Sample Location	LL	PL	PI	U.S.C.S. Classification
●	**B-1 at 35 ft**	**51**	**24**	**27**	**Fat CLAY with sand (CH)**
⧗	**B-1 at 50 ft**	**49**	**24**	**25**	**Lean CLAY (CL)**
▲	**B-2 at 25 ft**	**77**	**32**	**45**	**Fat CLAY (CH)**

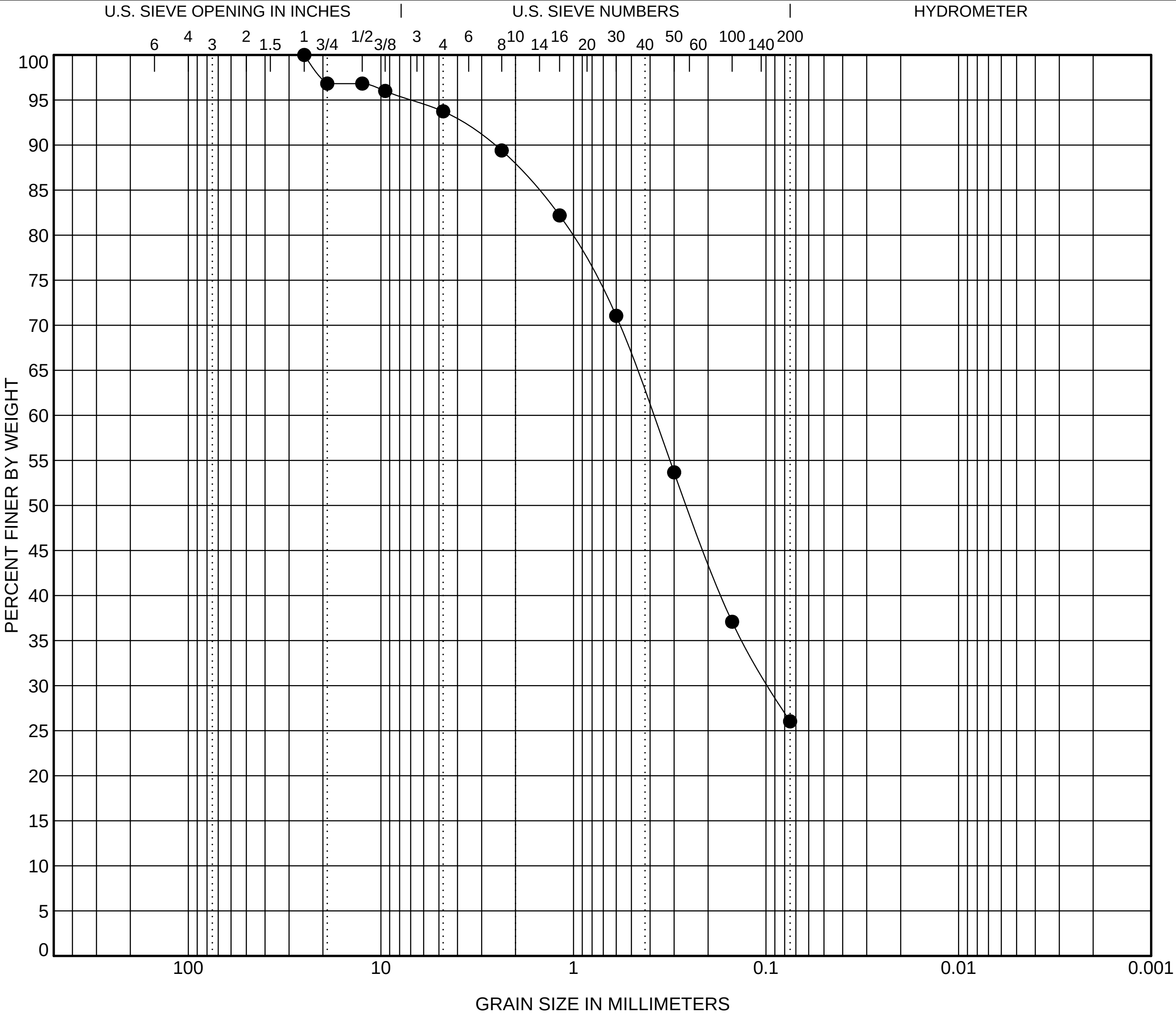
TWINING

ATTERBERG LIMITS

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE B-1

ATTERBERG LIMITS MAST PARK BORING LOGS.GPJ TWINING LABS.GDT 8/10/16



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-1 at 5 ft**	**Silty SAND (SM)**		

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
25	**0.386**	**0.257**	**0.096**		**6.3**	**67.7**	**26.0**	

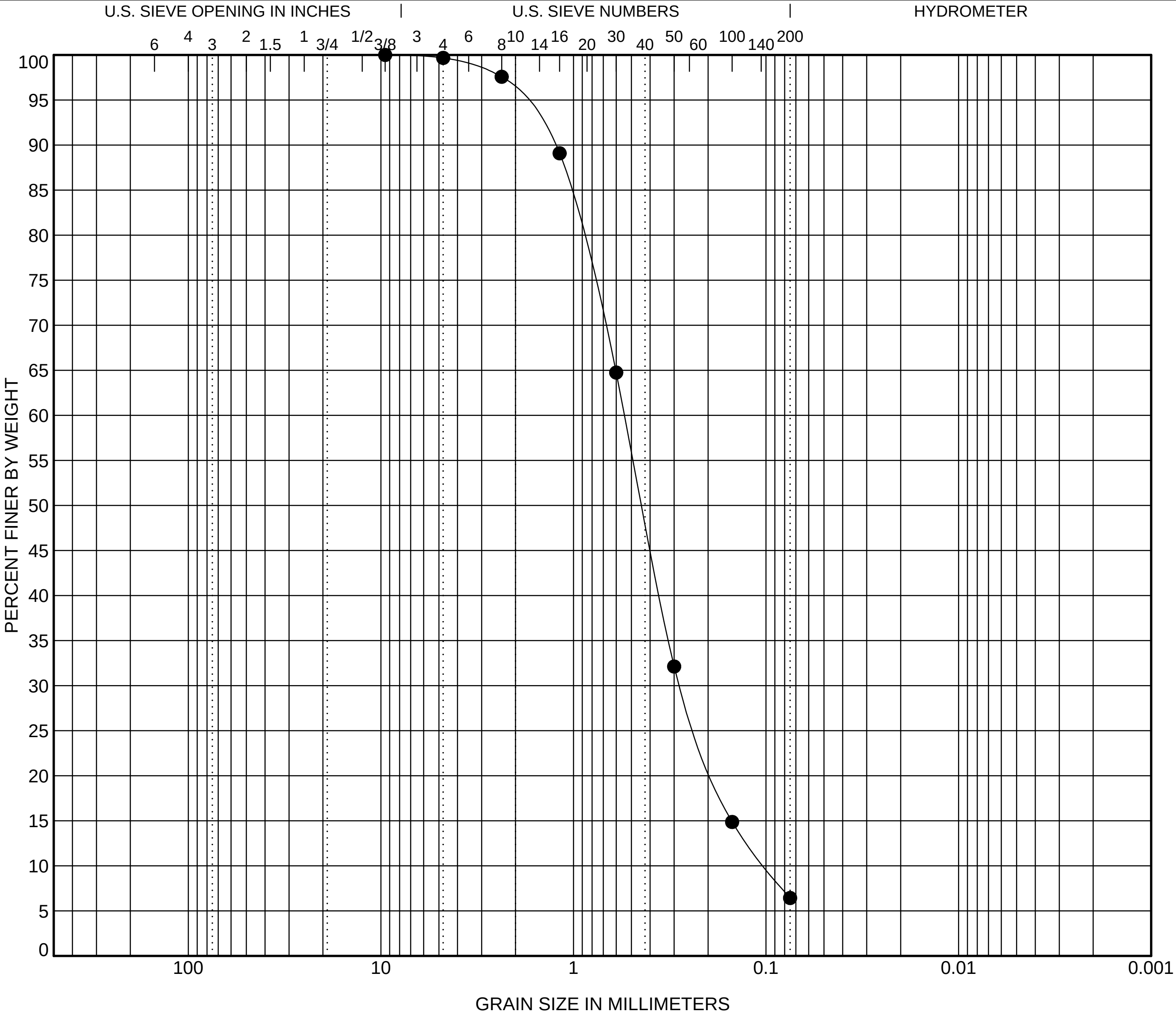
GRAIN SIZE MAST PARK BORING LOGS.GPJ TWINING LABS.GDT 8/10/16

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 2
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	U.S.C.S. Classification	Cc	Cu
●	**B-1 at 10 ft**	**Poorly graded SAND with silt (SP-SM)**	**1.39**	**5.39**

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
9.5	**0.542**	**0.439**	**0.275**	**0.101**	**0.3**	**93.2**	**6.4**	

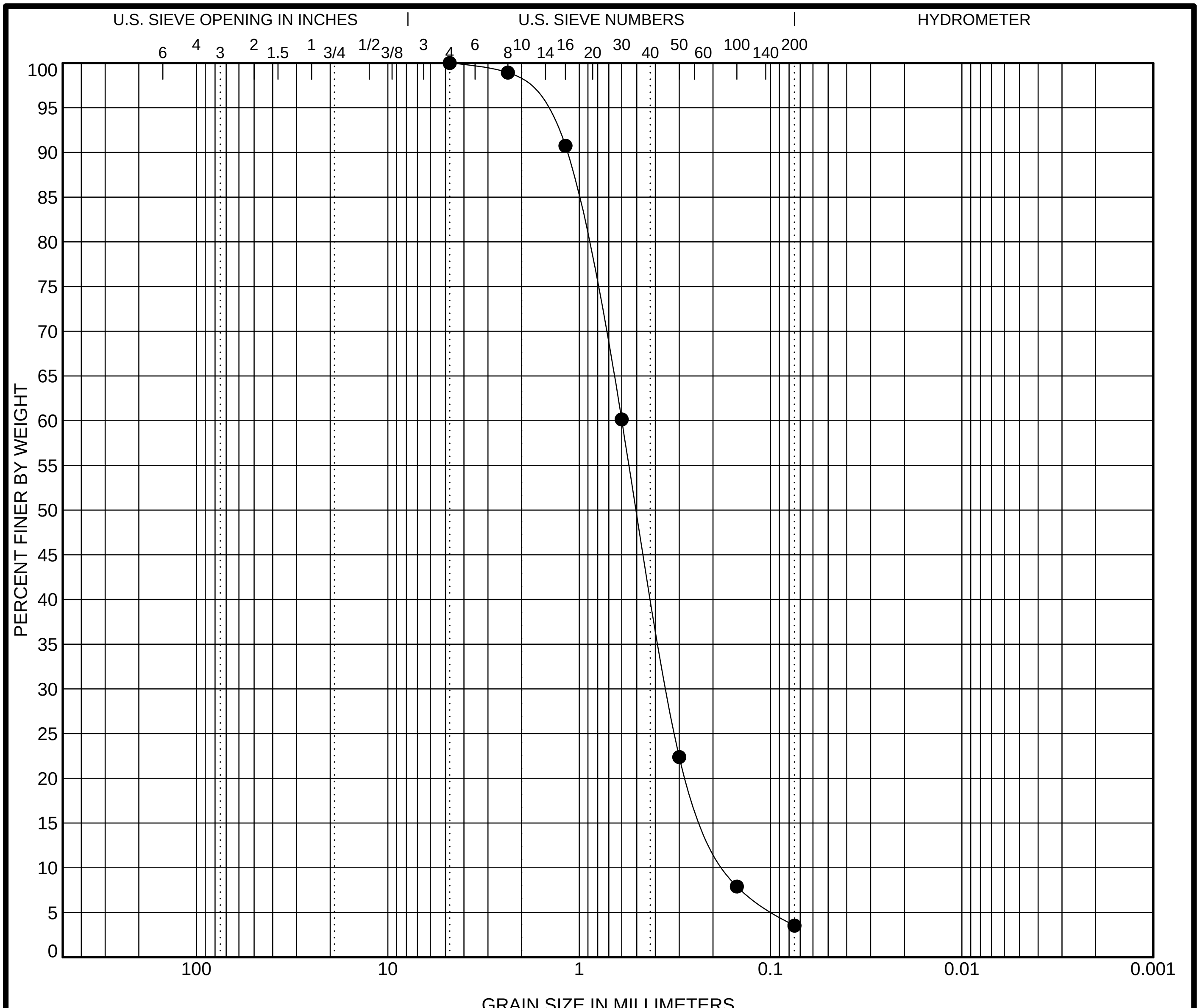
GRAIN SIZE MAST PARK BORING LOGS.GPJ TWINING LABS.GDT 8/10/16

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 3
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	U.S.C.S. Classification	Cc	Cu
●	**B-1 at 20 ft**	**Poorly graded SAND (SP)**	**1.20**	**3.61**

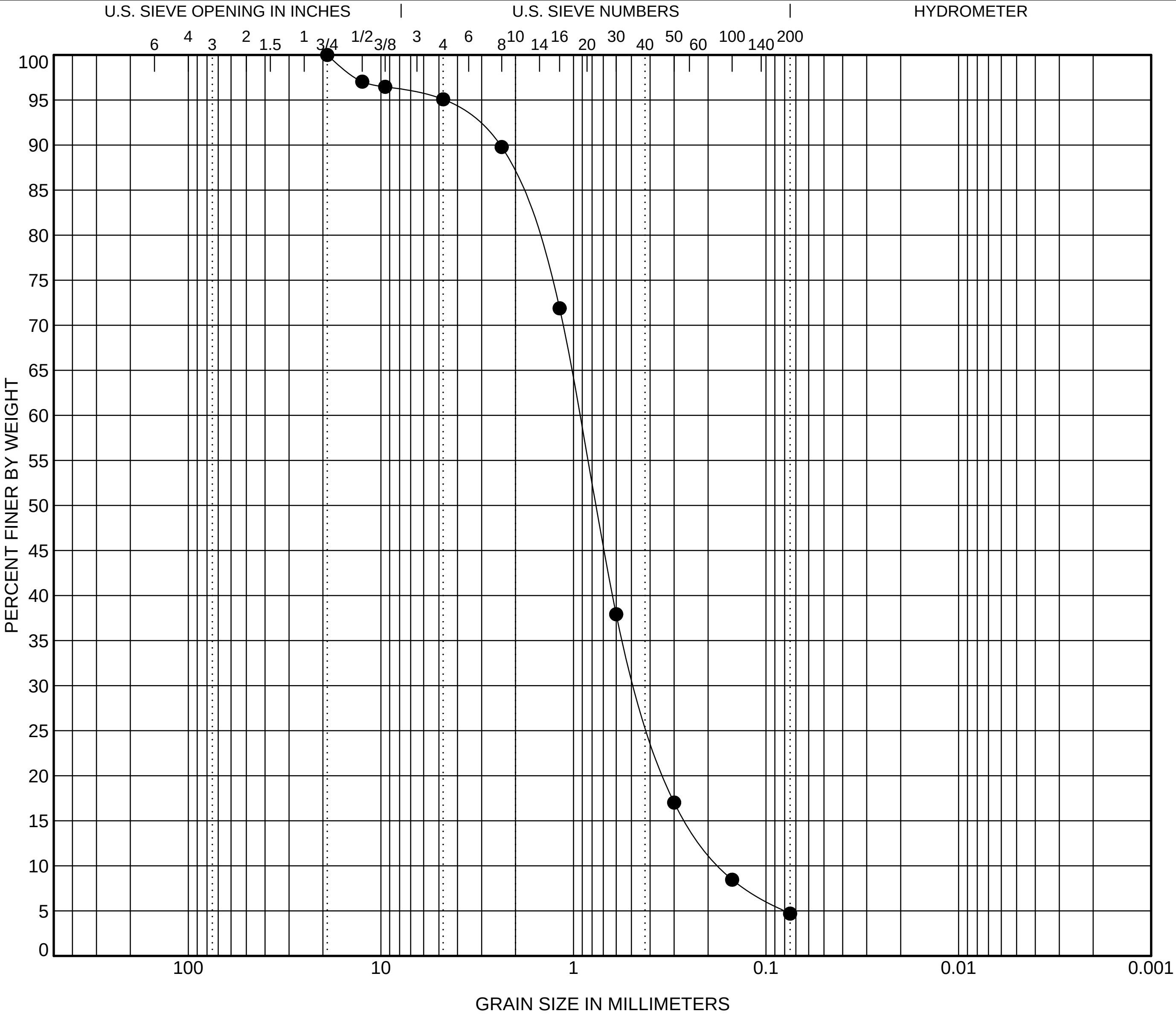
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
4.75	**0.598**	**0.498**	**0.345**	**0.166**	**0.0**	**96.5**	**3.5**	

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 4
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-1 at 30 ft**	**Poorly graded SAND (SP)**	**1.34**	**5.48**

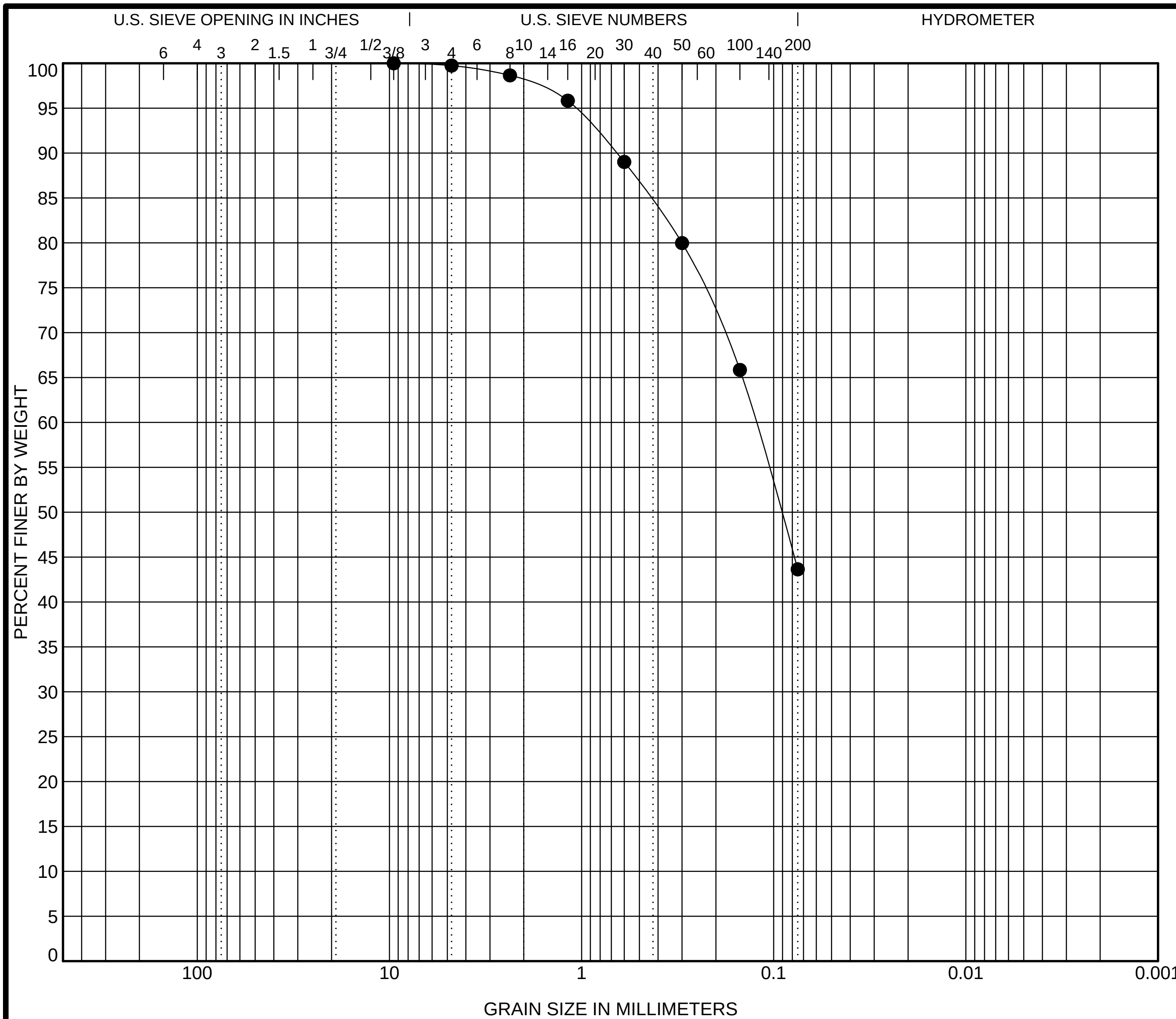
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
19	**0.931**	**0.763**	**0.461**	**0.17**	**4.9**	**90.4**	**4.7**	

GRAIN SIZE DISTRIBUTION

TWINING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 5
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-2 at 5 ft**	**Silty SAND (SM)**		

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
9.5	**0.125**	**0.091**			**0.3**	**56.1**	**43.6**	

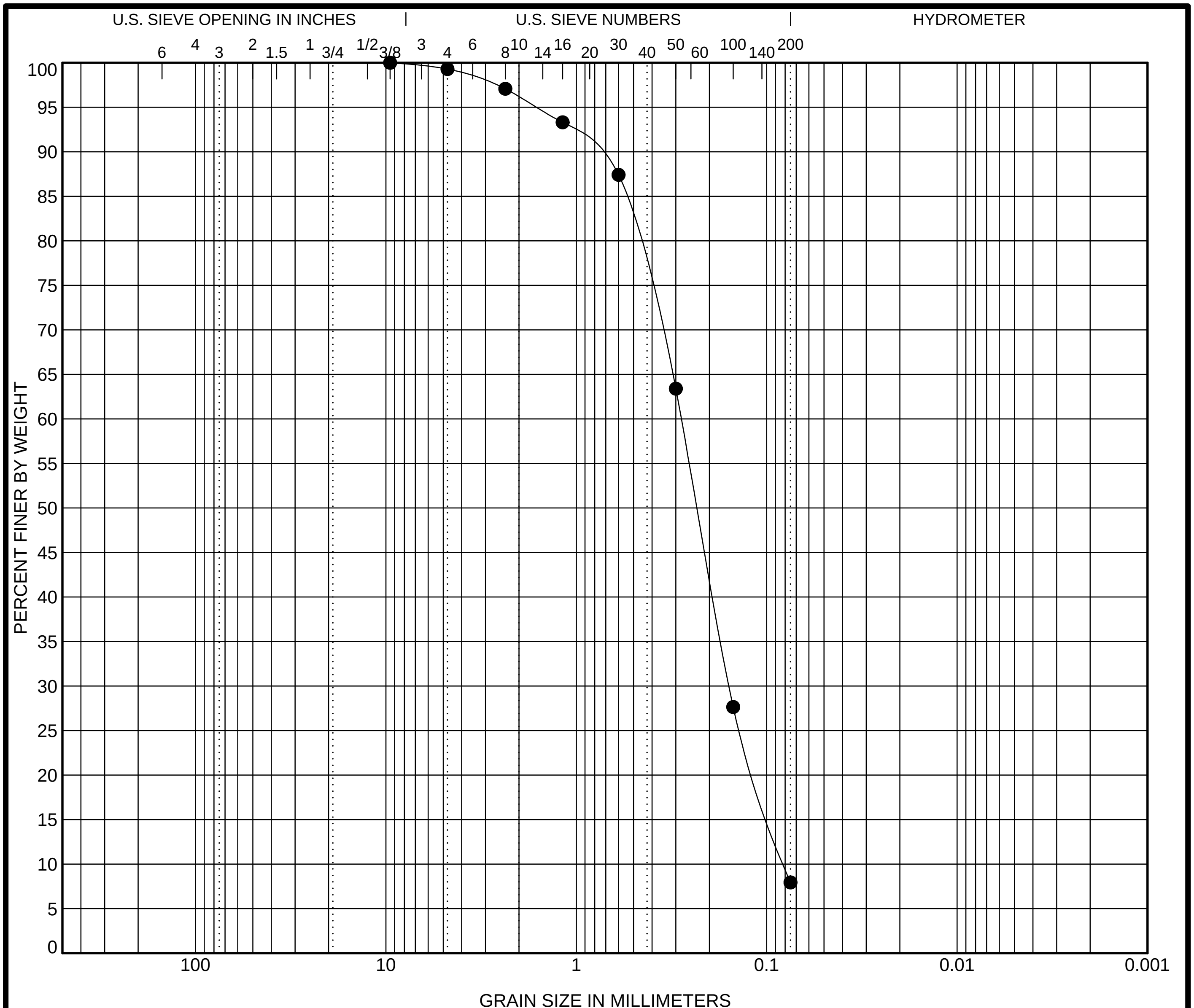
GRAIN SIZE MAST PARK BORING LOGS.GPJ TWINING LABS.GDT 8/10/16

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 6
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-5 at 0-5 ft**	**Poorly graded SAND with silt (SP-SM)**	**1.09**	**3.48**

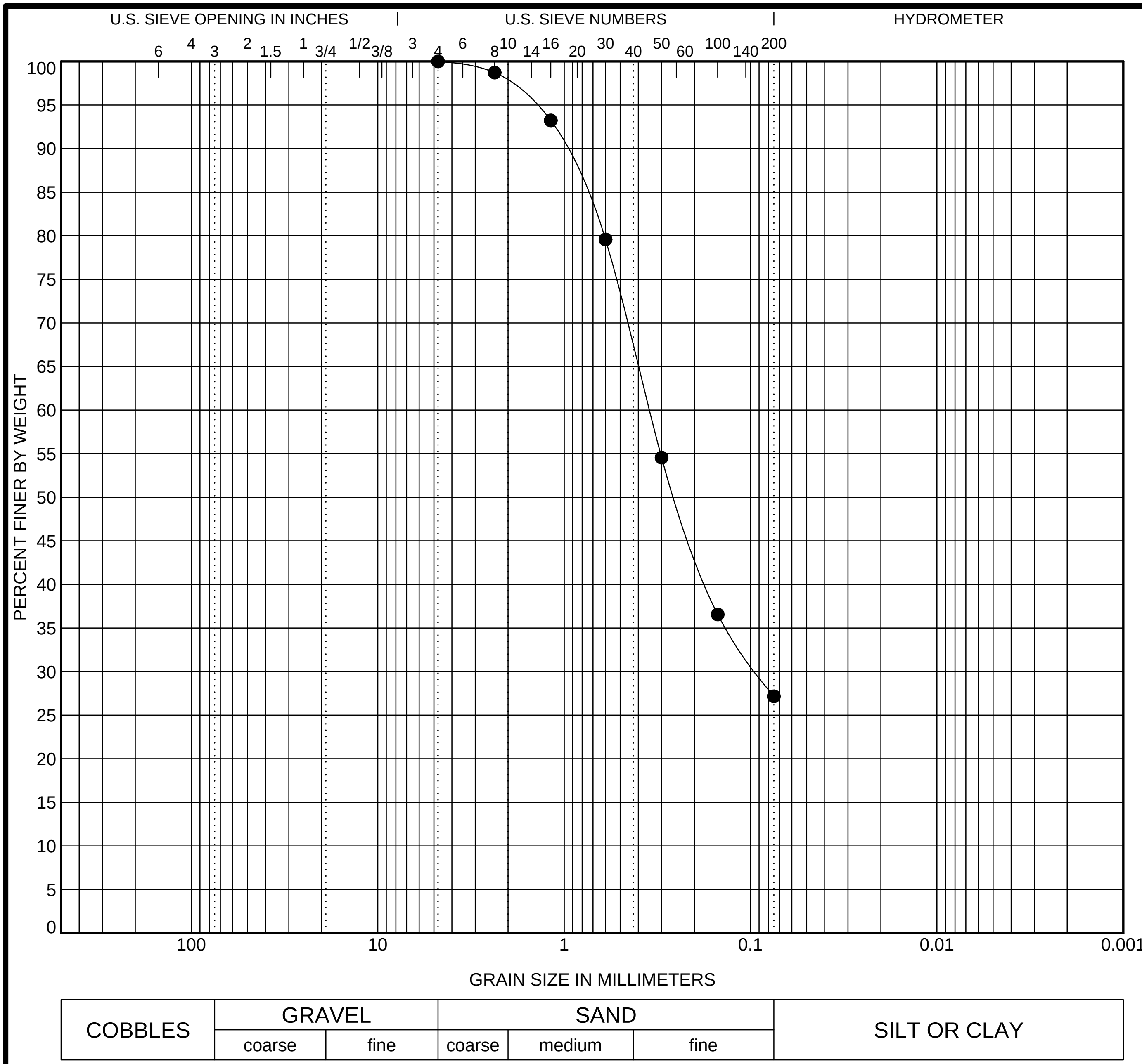
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
9.5	**0.281**	**0.231**	**0.157**	**0.081**	**0.7**	**91.4**	**7.9**	

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 7
160392.2	August 2016	



	Sample Location	U.S.C.S. Classification	Cc	Cu
●	**IF-1 at 0-3 ft**	**Silty SAND (SM)**		

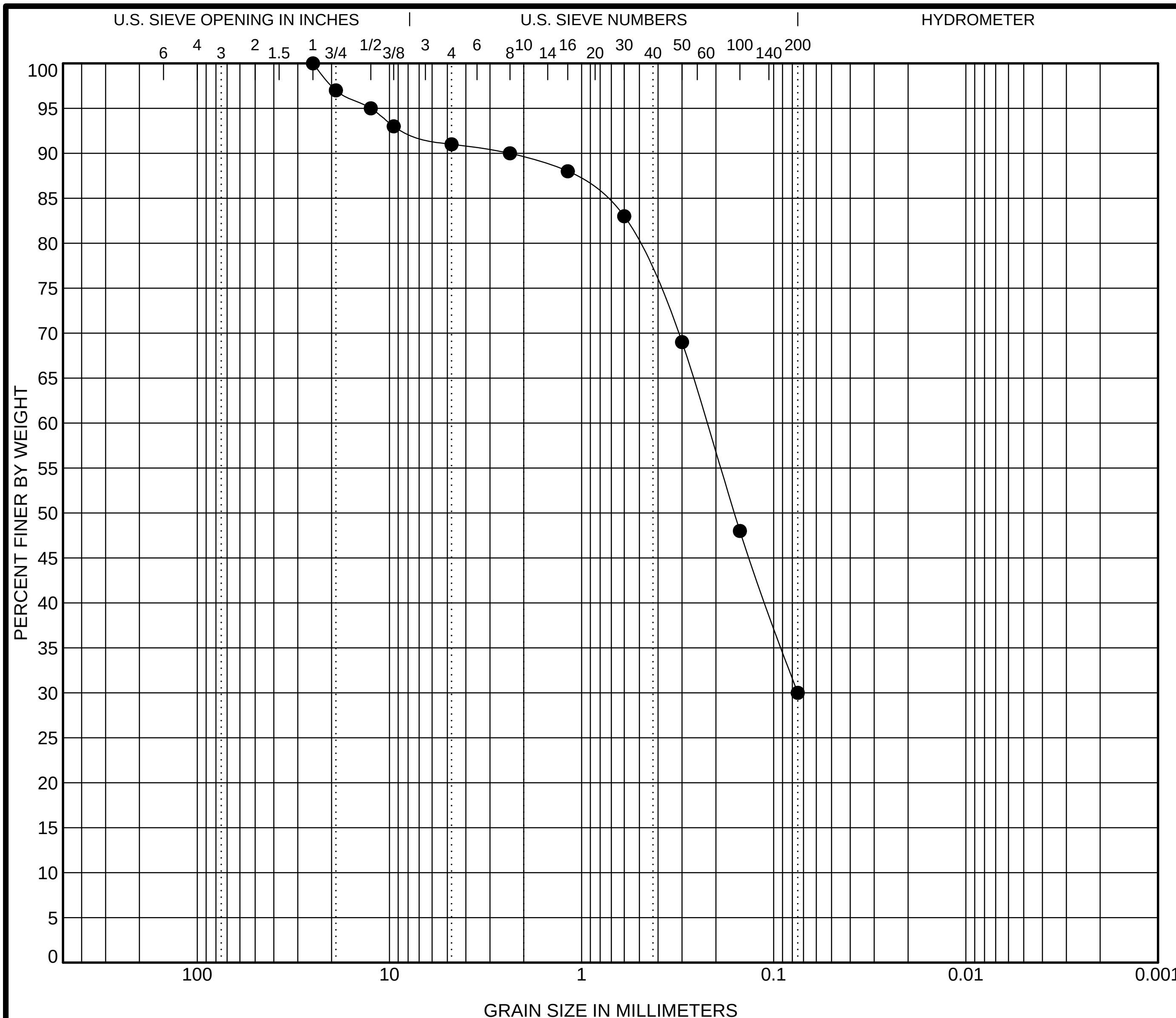
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
4.75	**0.349**	**0.252**	**0.092**		**0.0**	**72.8**	**27.2**	

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 8
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **IF-2 at 0-3 ft**	**Silty SAND (SM)**		

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
25	**0.223**	**0.16**	**0.075**		**9.0**	**61.0**	**30.0**	

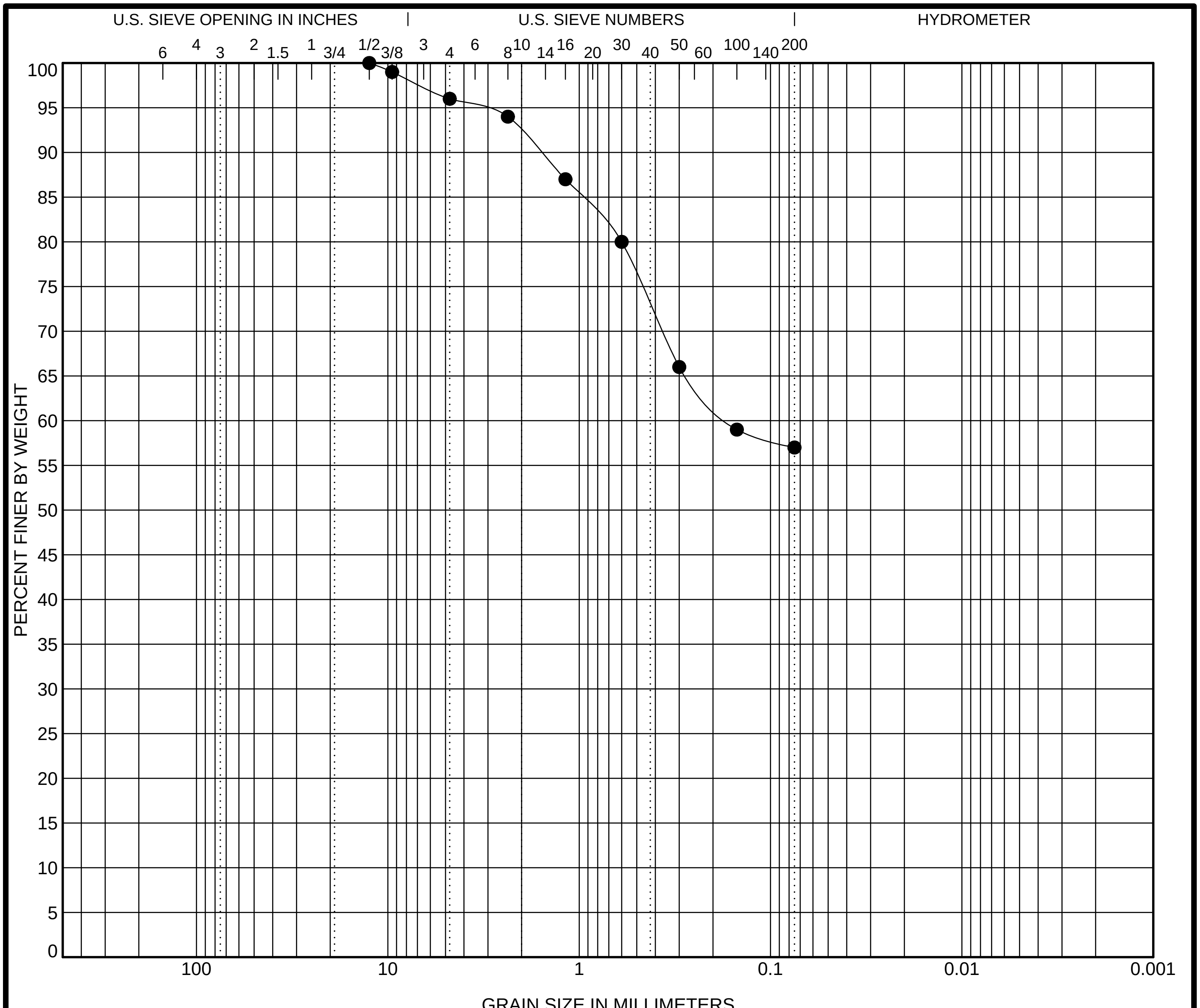
GRAIN SIZE MAST PARK BORING LOGS.GPJ TWINING LABS.GDT 8/10/16

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 9
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **IF-3 at 0-3 ft**	**Silty SAND (SM)**		

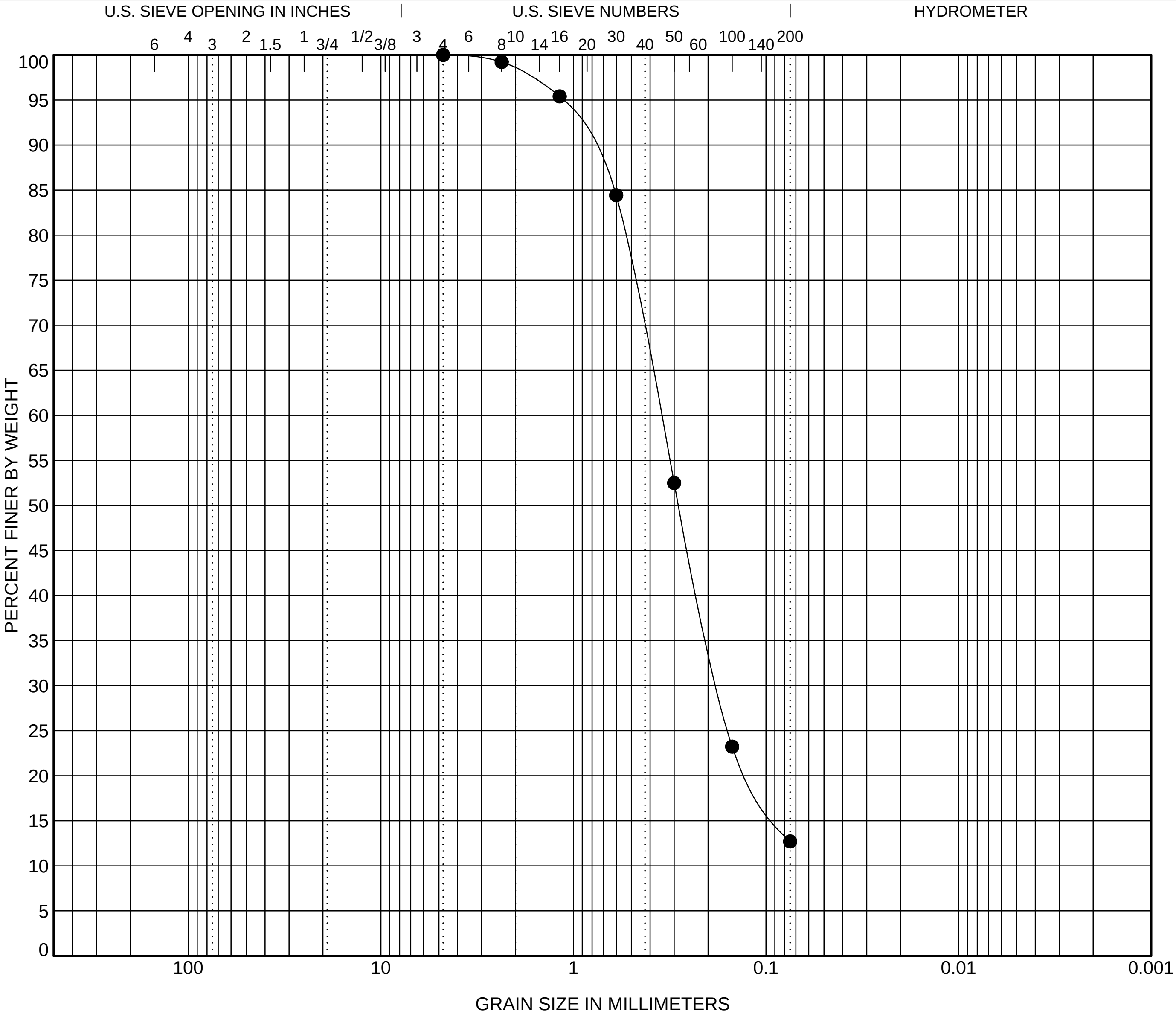
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
12.5	**0.166**				**4.0**	**39.0**	**57.0**	

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 10
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **IF-4 at 0-3 ft**	**Silty SAND (SM)**		

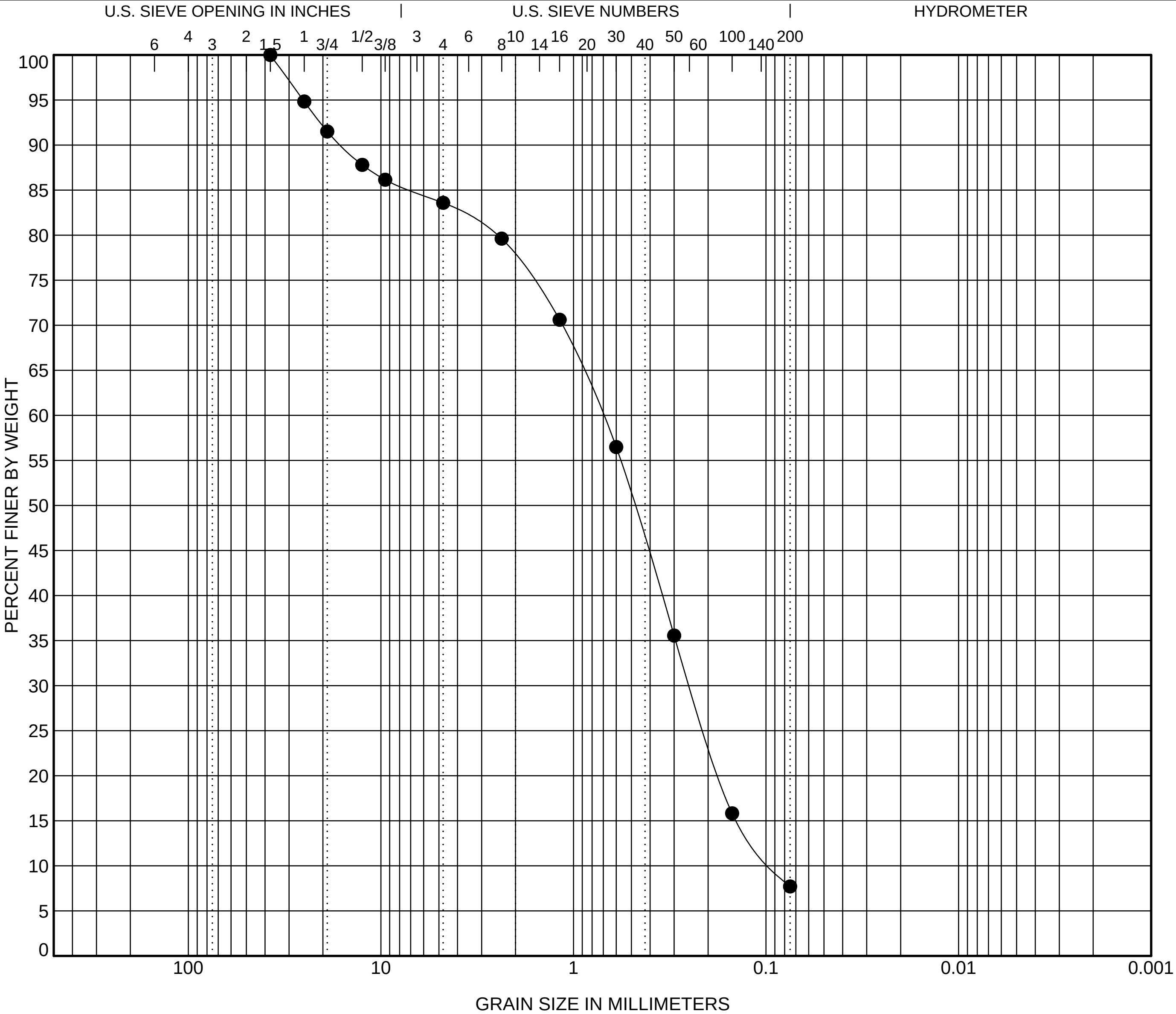
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
4.75	**0.353**	**0.283**	**0.176**		**0.0**	**87.3**	**12.7**	

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 11
160392.2	August 2016	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **IF-6A at 0-3 ft**	**Poorly graded SAND with silt (SP-SM)**	**0.94**	**7.79**

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
37.5	**0.71**	**0.484**	**0.247**	**0.091**	**16.4**	**75.9**	**7.7**	

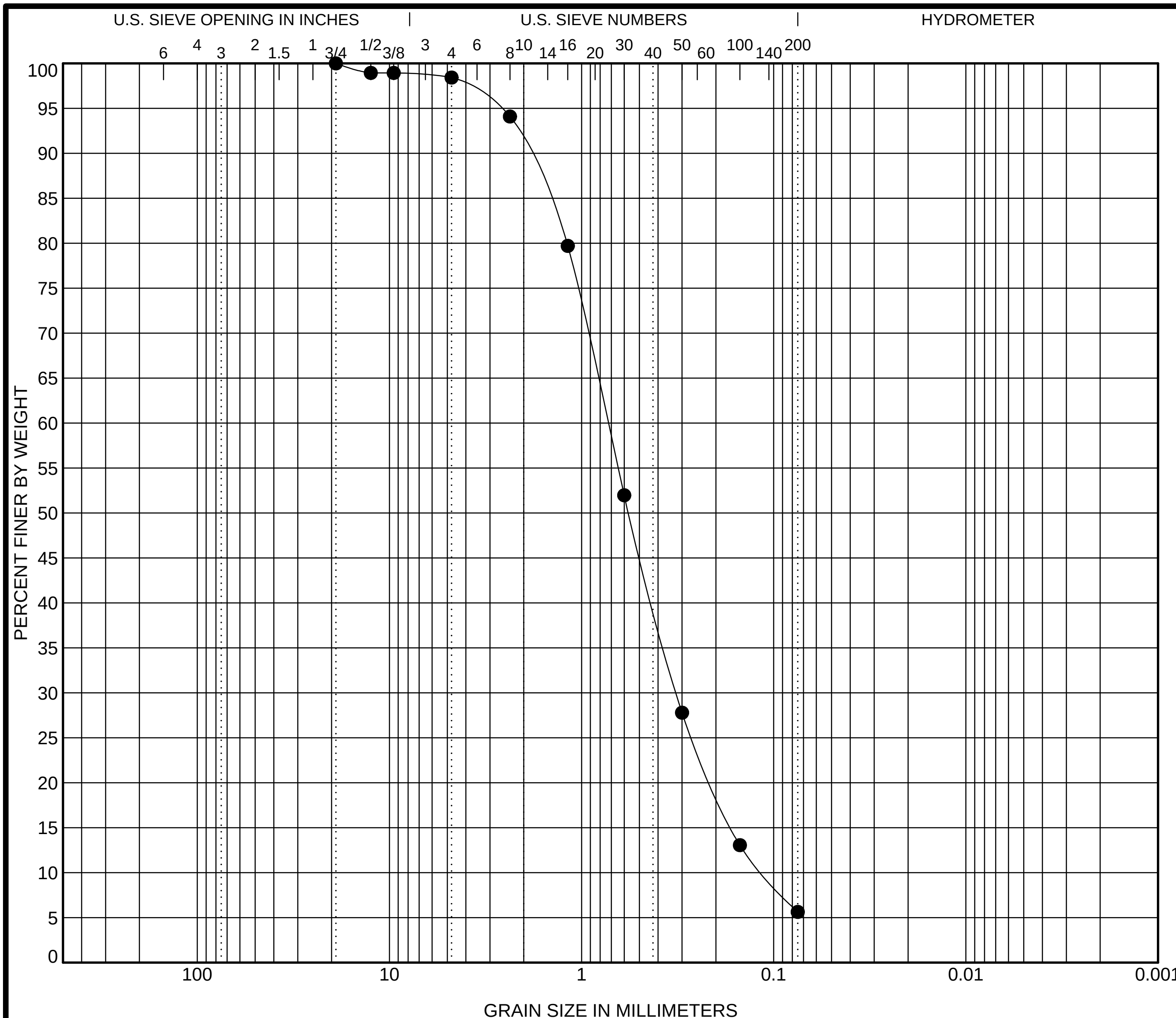
TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 12
160392.2	August 2016	

GRAIN SIZE MAST PARK BORING LOGS.GPJ TWINING LABS.GDT 8/10/16



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **IF-7 at 0-3 ft**	**Poorly graded SAND (SP)**	**1.24**	**6.47**

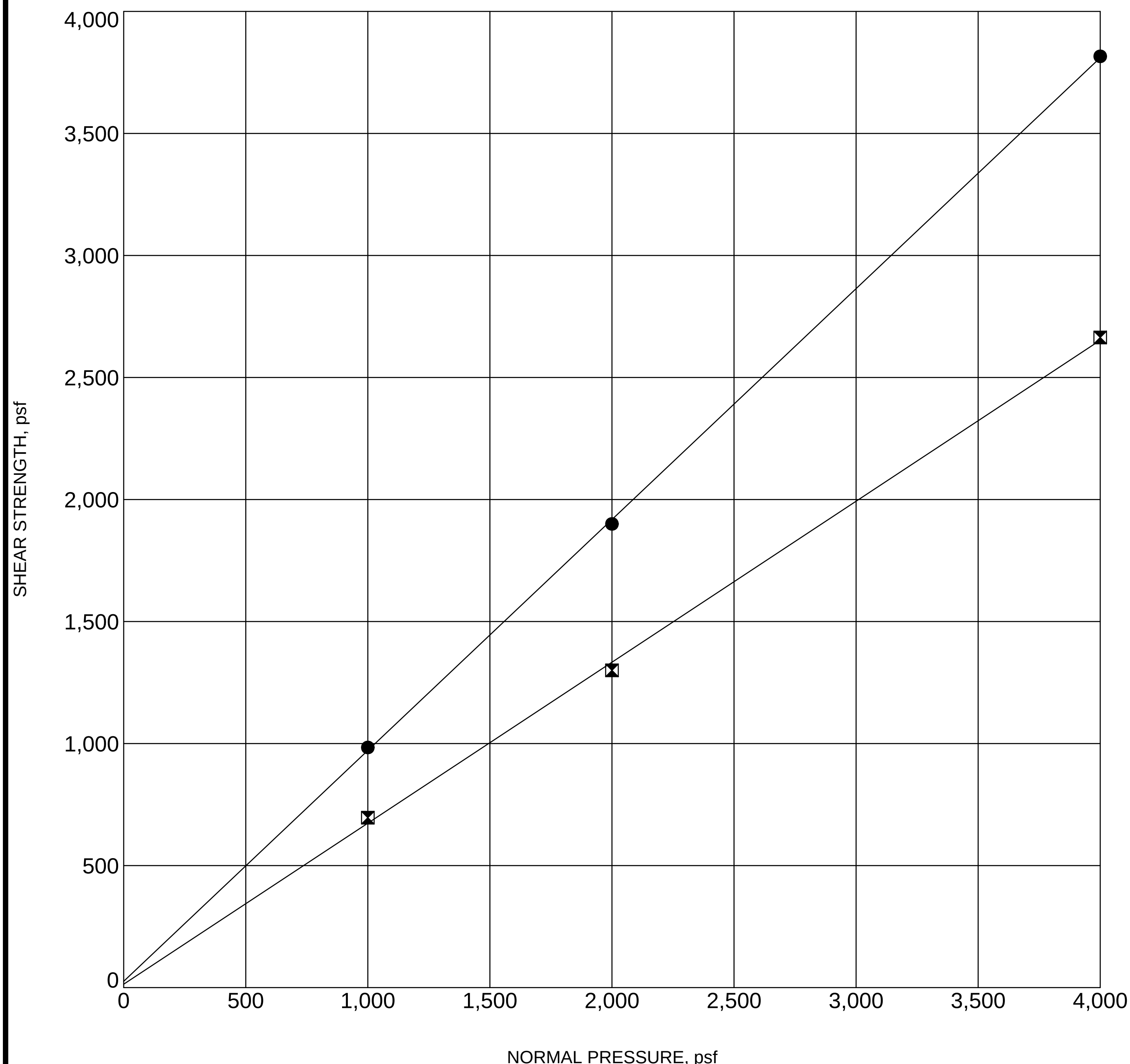
D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
19	**0.73**	**0.567**	**0.32**	**0.113**	**1.6**	**92.8**	**5.6**	

TWINING

GRAIN SIZE DISTRIBUTION

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B- 13
160392.2	August 2016	



Boring No.: B-1
Sample Depth (ft): 25
Sample Description: Poorly graded SAND (SP)
Strain Rate (in./min): 0.005
Dry Density (pcf): 111.9

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	25	15
Friction Angle, Ø (deg):	43	33

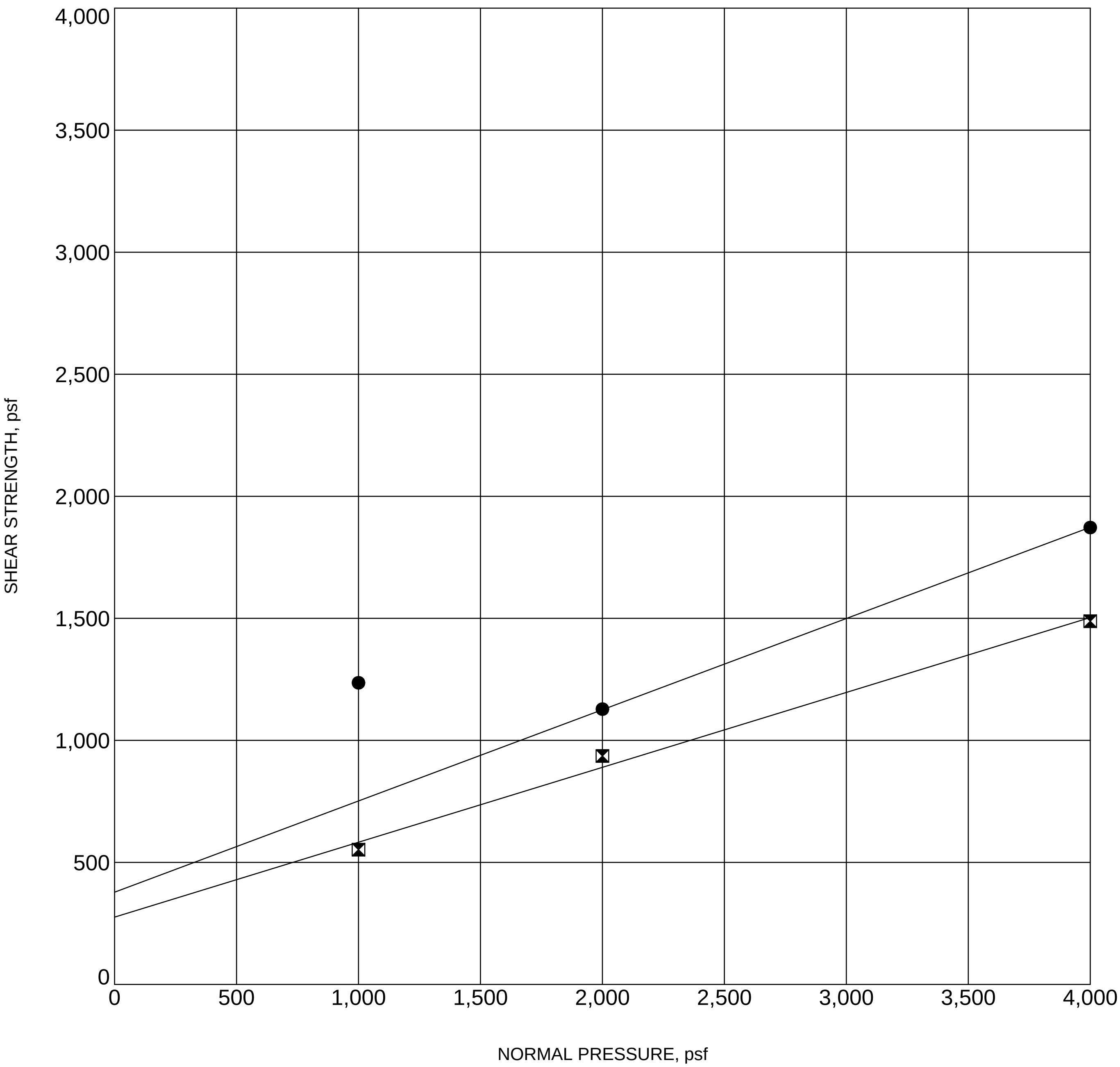
Initial Moisture (%): 15.9
Final Moisture (%): 19.8

TWINING

DIRECT SHEAR TEST

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	
160392.2	August 2016	FIGURE B-14



Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	380	275
Friction Angle, Ø (deg):	21	17

Initial Moisture (%): 18.2
Final Moisture (%): 22.9

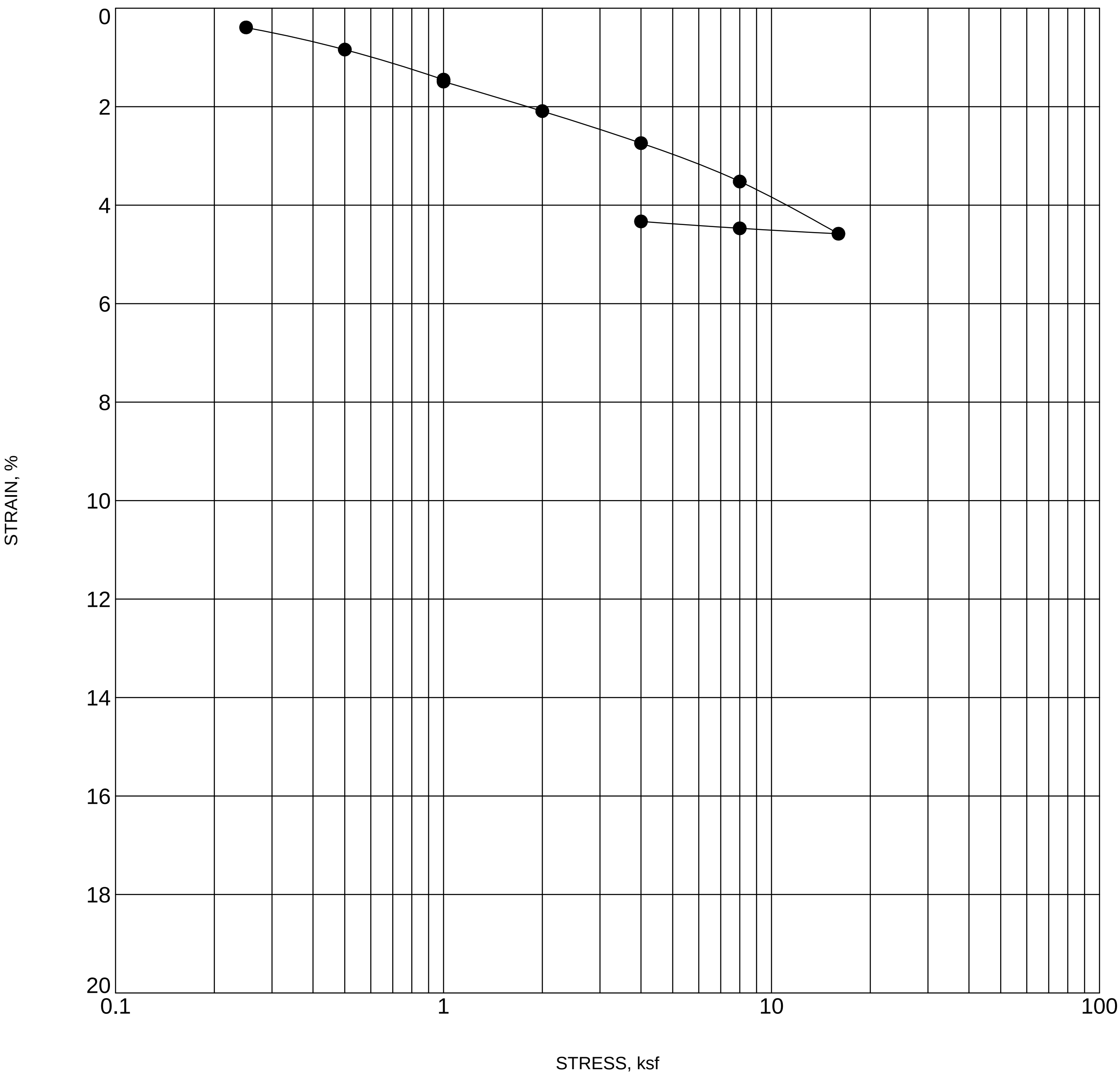
Boring No.: B-1
Sample Depth (ft): 45
Sample Description: Lean CLAY (CL)
Strain Rate (in./min): 0.005
Dry Density (pcf): 108.9

TWINING

DIRECT SHEAR TEST

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B-15
160392.2	August 2016	

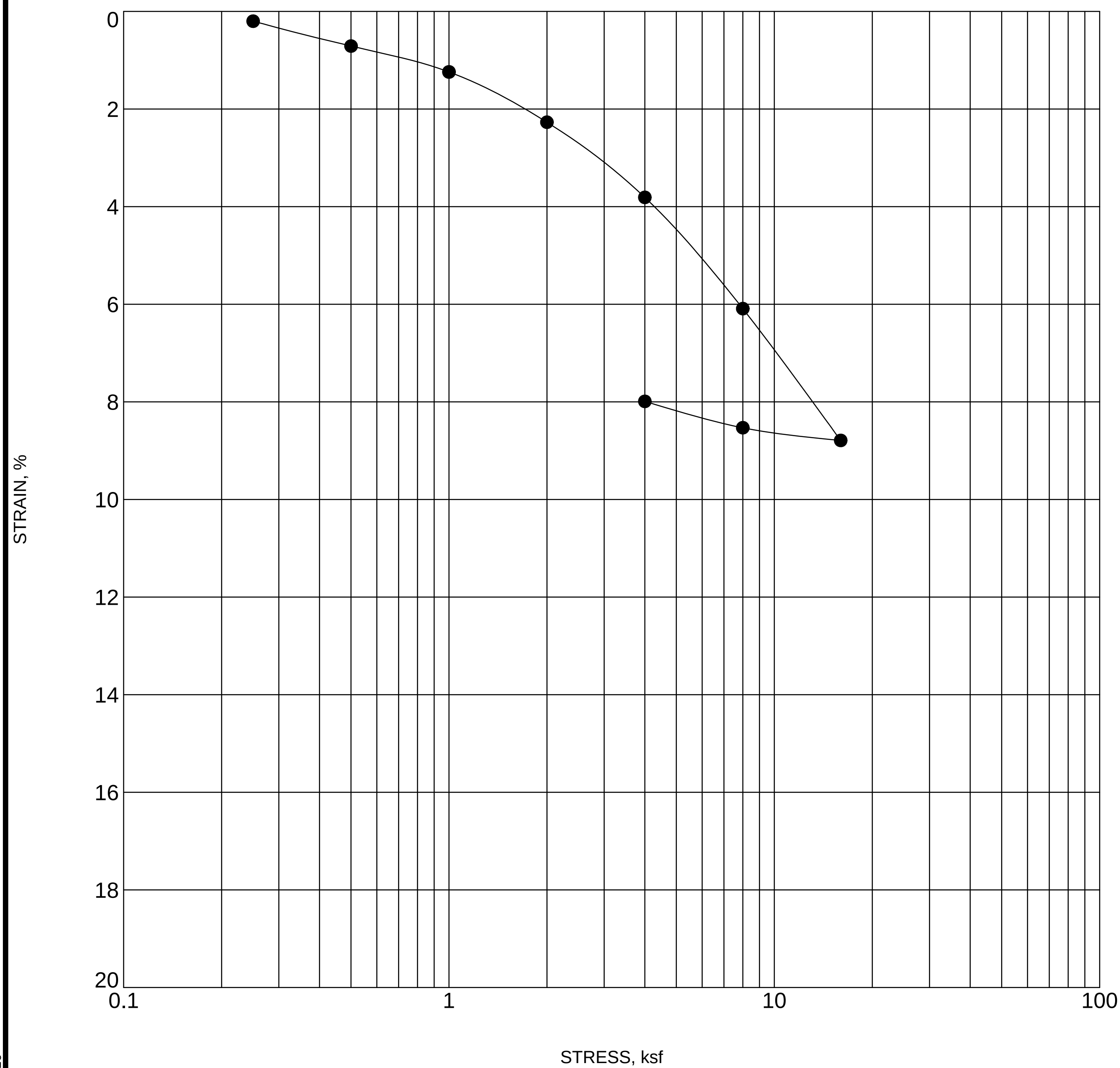


	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-1 at 15 ft**	**Poorly graded SAND with silt (SP-SM)**	**106.6**	**16.9**

CONSOLIDATION TEST

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B-16
160392.2	August 2016	



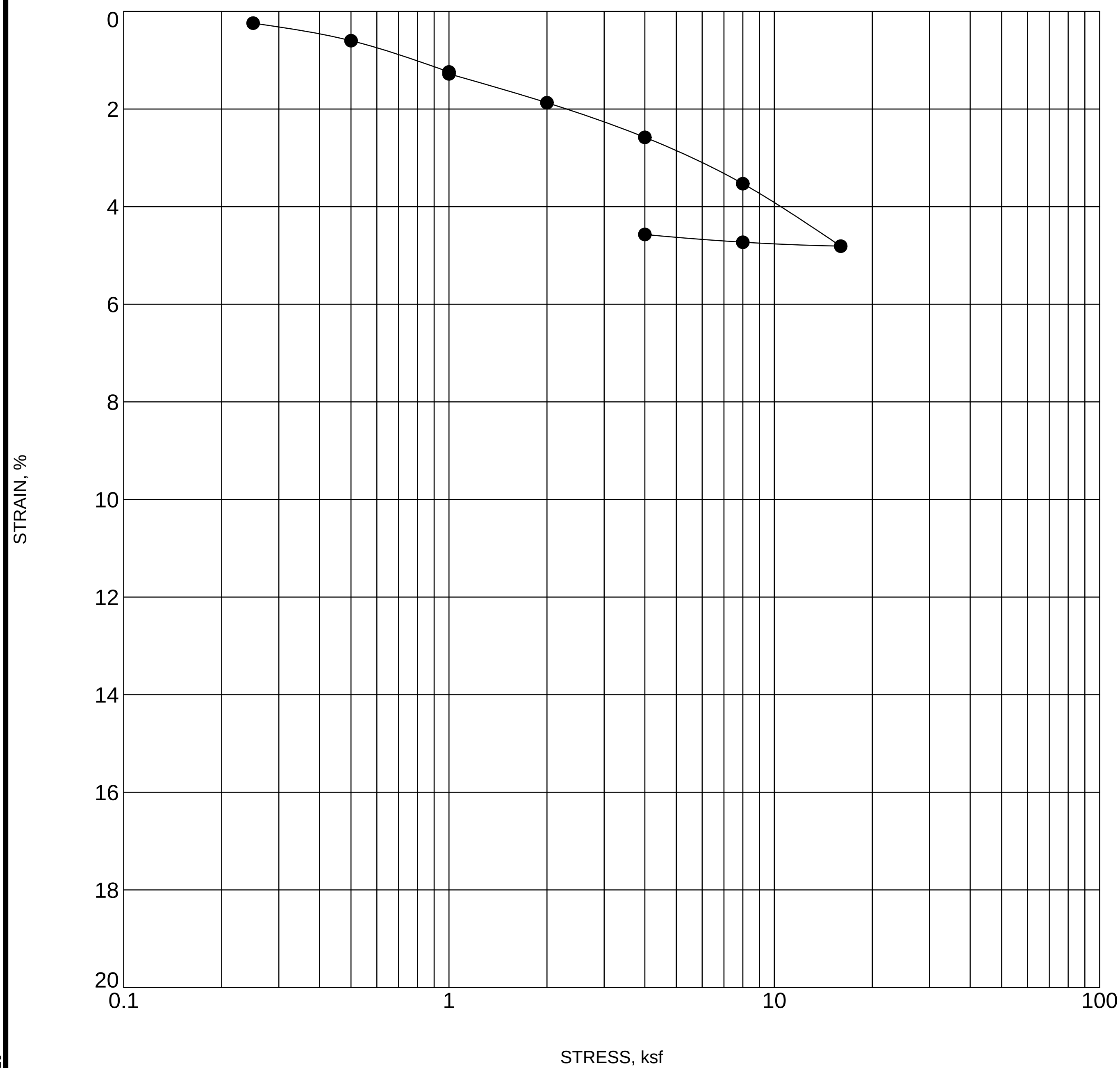
Sample Location		Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-1 at 35 ft**	**Fat CLAY with sand (CH)**	**96.9**	**20.6**

CONSOLIDATION TEST

TWINING

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B-17
160392.2	August 2016	



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-2 at 10 ft**	**Poorly graded SAND (SP)**	**107.2**	**12.1**

TWINING

CONSOLIDATION TEST

Mast Park Improvement Project
Santee, California

PROJECT NO.	REPORT DATE	FIGURE B-18
160392.2	August 2016	

APPENDIX C
PERCOLATION TESTS

TWINING CONSULTING
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July 22, 2016
Project No. 160392.2

Mr. Mark Tarrall
Dokken Engineering
5675 Ruffin Rd # 250
San Diego, California 92123

Subject: Preliminary Infiltration Rate Report
Mast Park Improvements Project
Santee, California

References: City of Santee BMP Design Manual, dated February 2016.

Schmidt Design Group, Mast Park Master Plan Report, dated January 3, 2012.

Dear Mr. Tarrall:

Twining, Inc., is pleased to present the preliminary results of borehole percolation testing for the subject project. Percolation testing was performed at seven locations within the Mast Park study area located east of Carlton Hills Blvd, north of the San Diego River and west and south of existing residential developments in the City of Santee, California. The study area covers approximately 30 acres. Percolation testing locations and depths were provided to us by Dokken Engineering and are presented on the attached Figure 2, Exploration Location Map. The purpose of our testing was to evaluate design infiltration rates of site subgrade soils and determine the feasibility of implementing water quality best management practices (BMP).

FIELD EXPLORATION

On June 30, 2016 we excavated seven percolation test borings using a truck mounted drill rig with an 8-inch diameter auger. All borings were excavated to a depth of 3 feet below existing ground surface (bgs) except for boring IF-5 which was excavated to a depth of 5 feet bgs as requested by Dokken Engineering. The borings exposed damp to moist, medium brown, silty sand, sand with silt, and lean clay with gravel which is consistent with the materials associated with alluvial deposits as described in the regional geologic map for the site. Soil samples were obtained at each location for laboratory testing. The sieve analysis test results are attached in Appendix A.

Groundwater was not encountered during percolation testing except for boring IF-5 where we observed groundwater at a depth of 4½ feet bgs. A test boring was also excavated near boring IF-6 and groundwater was observed at a depth of 7 feet bgs. It is important to note that according to the referenced City of Santee BMP Design Manual (2016), the suitability assessment for infiltration facility bottoms located within 5 feet to 15 feet of the groundwater level is of medium concern which will affect the factor of safety used for design.

PERCOLATION TESTING

Borehole percolation testing was performed in general conformance with the referenced City of Santee BMP Design Manual (2016). As indicated above, borings IF-1 to IF-4, IF6 and IF 7 had an approximate diameter of 8-inches and were excavated to a depth of 3 feet depth bgs. Boring IF-5 was excavated to a depth of 5 feet bgs. The percolation test at location IF-5 was not performed due to the presence of groundwater at a depth of 4½ feet bgs.

At the completion of excavation, approximately 2 inches of coarse gravel was placed at the bottom of the boreholes to prevent scouring during testing. Perforated PVC pipe sections were inserted in the boreholes and coarse gravel was used as backfill around the pipes. The boreholes were presoaked prior to testing during two 30-minute intervals in accordance with City of Santee BMP Design Manual guidelines for sandy soil materials.

On July 1, the borings were filled with water. Water level drop measurements were taken at minimum 10-minute intervals for at least 6 readings. After a stable reading was observed, the drop that occurred during the final reading was used to determine the percolation rate at each test location.

The following conversion equation was used for the final reading to calculate the infiltration rate:

Infiltration Rate = $I_t = \Delta H(60r) / [\Delta t(r + 2H_{avg})]$, where:

Δt = time interval (in minutes)
r = test hole radius = d/2
D_0 = initial depth to water
D_f = final depth to water
D_T = total depth of test hole
H_0 = initial height of water at the selected time interval = $D_T - D_0$
H_f = final height of water at the selected time interval = $D_T - D_f$
$\Delta H = \Delta D$ = change in height over the time interval = $H_0 - H_f$
$H_{avg} = (H_0 + H_f) / 2$

The design infiltration rate was calculated from the measured infiltration by applying a factor of safety (FS) of 4.375, as required by the referenced manual. The assumptions made to obtain the factor of safety may be revised after discussion Dokken Engineering. A summary of the test results is presented in Table 1. Additional test details are presented in the attached percolation datasheets.

Table 1 - Summary of Percolation Test Results

Test Location	Depth of Test Hole (in.)	Soil Type	Measured Infiltration Rate (in/hr)	Design Infiltration Rate (in/hr)
IF-1	36	Silty Sand	0.63	0.14
IF-2	36	Silty Sand	2.57	0.59
IF-3	36	Sandy lean Clay	0.01	0.003
IF-4	36	Silty Sand	3.15	0.72
IF-5	60	Sand with Silt	Abandoned	
IF-6	36	Sand with Silt and Gravel	0.98	0.22
IF-7	36	Sand with Silt	30.3	6.9

RECOMMENDATIONS

Based on the results of our testing and analyses, the installation of the proposed water quality BMPs is feasible. However, the infiltration rate at the site is highly variable and additional testing is recommended at the specific BMP location prior to final design. It is also recommended that the proposed water quality BMPs comply with the setback requirements presented in Table 2.

Table 2: Recommended Infiltration Facility Setback Requirements

Setback from	Distance
Property lines and public right of way	5 feet
Any foundation	15 feet or within 1:1 plane drawn up from the bottom of foundation, whichever is greater
Face of any slope	H/2, 5 feet minimum (H is height of slope)
Water wells used for drinking water	100 feet

LIMITATIONS

Due to the limited nature of our field exploration, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during construction.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or that activities of man at the subject site or at nearby sites. Changes to applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Twining, Inc. has no control.

We have endeavored to perform our evaluation using the degree of care and skill ordinarily exercised under similar circumstances by engineering professionals with experience in this area. No other warranty, either expressed or implied, is made as to the conclusions contained in this report.

We appreciate the opportunity to be of service on this project. If you have any questions regarding this report, or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
TWINING, INC.

REGISTERED PROFESSIONAL ENGINEER
ANDRES BERNAL
No. 2715
Exp. 9/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

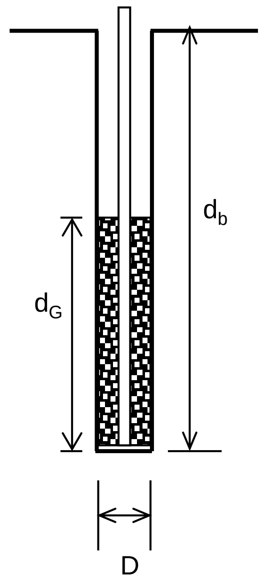
Andres Bernal, RCE 62366, GE 2715
Senior Geotechnical Engineer

Adrian Moreno, EIT
Staff Engineer

Attachments: Percolation Test Data
Worksheet D.5-1: Factor of Safety and Design Infiltration Rate Worksheet
Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

PERCOLATION TEST DATA

Project No.: 160392.2
Project Name: Mast Park Improvements
Test Date: June 30, 2016
Test Boring No.: **IF-1**
Diameter of Boring (D): 8.0 inch
Depth of Boring (d_b): 36.0 inch
Performed by: SM



Sandy Soil Criteria Test

Time of Testing			Water Level Measurements			Greater than or Equal to 6"?
Start Time T_i	Stop Time T_f	Time Interval (min) ΔT	Initial depth to water (inch) d_1	Final depth to water (inch) d_2	Drop of water column (inch) $\Delta d = d_i - d_f$	(Yes/No)
11:15 AM	11:45 AM	30	28.00	32.00	4.00	No
11:48 AM	12:18 PM	30	28.00	31.50	3.50	No

Factor of Safety (FS) = 4.375

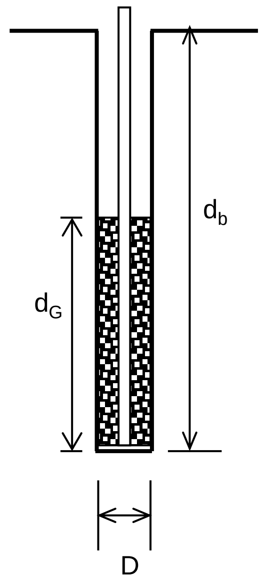
Time of Testing			Water Level Measurements		Water Level Calculations			Infiltration Rate Calculations	
Start Time T_i	Stop Time T_f	Time Interval ΔT (min)	Initial depth to water d_1 (inch)	Final depth to water d_2 (inch)	Initial height of water column d_i (inch)	Final height of water column d_f (inch)	Drop of water column $\Delta d = d_i - d_f$ (inches)	Tested Infiltration Rate ***It*** (inch/hr)	Design Infiltration Rate ***It/FS*** (inch/hr)
Percolation Test									
12:29 PM	1:04 PM	35	28.00	30.00	8.00	6.00	2.00	0.76	0.17
1:05 PM	1:37 PM	32	28.00	29.75	8.00	6.25	1.75	0.72	0.16
1:39 PM	2:11 PM	32	28.00	29.63	8.00	6.38	1.63	0.66	0.15
2:14 PM	2:49 PM	35	28.00	29.63	8.00	6.38	1.63	0.61	0.14
2:50 PM	3:22 PM	32	28.00	29.56	8.00	6.44	1.56	0.64	0.15
3:23 PM	3:54 PM	31	28.00	29.50	8.00	6.50	1.50	0.63	0.14

	It	*It/FS*
Infiltration Rate (inch/hr)*:	**0.63**	**0.14**

Reference: City of Santee BMP Design Manual (2016)
*Based on the drop measured in the final reading.

PERCOLATION TEST DATA

Project No.: 160392.2
Project Name: Mast Park Improvements
Test Date: June 30, 2016
Test Boring No.: **IF-2**
Diameter of Boring (D): 8.0 inch
Depth of Boring (d_b): 36.0 inch
Performed by: SM



Sandy Soil Criteria Test

Time of Testing			Water Level Measurements			Greater than or Equal to 6"?
Start Time T_i	Stop Time T_f	Time Interval (min) ΔT	Initial depth to water (inch) d_1	Final depth to water (inch) d_2	Drop of water column (inch) $\Delta d = d_i - d_f$	(Yes/No)
10:30 AM	11:00 AM	30	28.00	36.00	8.00	Yes
11:05 AM	11:35 AM	30	28.00	36.00	8.00	Yes

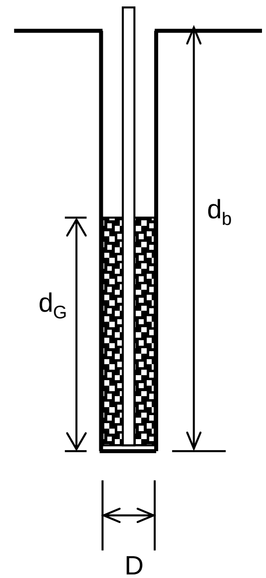
Factor of Safety (FS) = 4.375

Time of Testing			Water Level Measurements		Water Level Calculations			Infiltration Rate Calculations	
Start Time T_i	Stop Time T_f	Time Interval ΔT (min)	Initial depth to water d_1 (inch)	Final depth to water d_2 (inch)	Initial height of water column d_i (inch)	Final height of water column d_f (inch)	Drop of water column $\Delta d = d_i - d_f$ (inches)	Tested Infiltration Rate ***It*** (inch/hr)	Design Infiltration Rate ***It/FS*** (inch/hr)
Percolation Test									
11:50 AM	12:00 PM	10	27.00	31.50	9.00	4.50	4.50	6.17	1.41
12:02 PM	12:12 PM	10	27.00	31.00	9.00	5.00	4.00	5.33	1.22
12:15 PM	12:25 PM	10	27.00	30.50	9.00	5.50	3.50	4.54	1.04
12:28 PM	12:38 PM	10	27.00	29.50	9.00	6.50	2.50	3.08	0.70
12:39 PM	12:49 PM	10	27.00	29.38	9.00	6.63	2.38	2.90	0.66
12:50 PM	1:00 PM	10	27.00	29.25	9.00	6.75	2.25	2.73	0.62
1:01 PM	1:11 PM	10	27.00	29.19	9.00	6.81	2.19	2.65	0.61
1:13 PM	1:23 PM	10	27.00	29.13	9.00	6.88	2.13	2.57	0.59
								It	***It/FS***
							Infiltration Rate (inch/hr)*:	**2.57**	**0.59**

Reference: City of Santee BMP Design Manual (2016)
*Based on the drop measured in the final reading.

PERCOLATION TEST DATA

Project No.: 160392.2
Project Name: Mast Park Improvements
Test Date: June 30, 2016
Test Boring No.: **IF-3**
Diameter of Boring (D): 8.0 inch
Depth of Boring (d_b): 36.0 inch
Performed by: SM



Sandy Soil Criteria Test

Time of Testing			Water Level Measurements			Greater than or Equal to 6"?
Start Time T_i	Stop Time T_f	Time Interval (min) ΔT	Initial depth to water (inch) d_1	Final depth to water (inch) d_2	Drop of water column (inch) $\Delta d = d_i - d_f$	(Yes/No)
10:30 AM	11:00 AM	30	28.00	36.00	8.00	Yes
11:05 AM	11:35 AM	30	28.00	36.00	8.00	Yes

Factor of Safety (FS) = 4.375

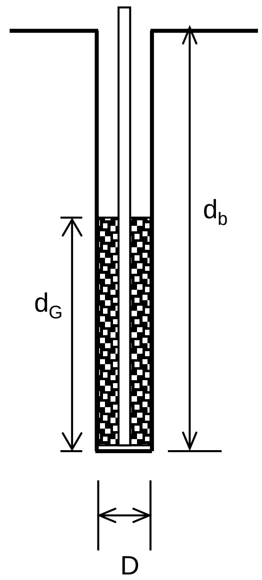
Time of Testing			Water Level Measurements		Water Level Calculations			Infiltration Rate Calculations	
Start Time T_i	Stop Time T_f	Time Interval ΔT (min)	Initial depth to water d_1 (inch)	Final depth to water d_2 (inch)	Initial height of water column d_i (inch)	Final height of water column d_f (inch)	Drop of water column $\Delta d = d_i - d_f$ (inches)	Tested Infiltration Rate ***It*** (inch/hr)	Design Infiltration Rate ***It/FS*** (inch/hr)
Percolation Test									
9:45 AM	10:45 AM	60	28.00	28.50	8.00	7.50	0.50	0.10	0.023
10:45 AM	11:45 AM	60	28.00	28.25	8.00	7.75	0.25	0.05	0.012
11:46 AM	12:46 PM	60	28.00	28.13	8.00	7.88	0.13	0.03	0.006
12:48 PM	1:48 PM	60	28.00	28.06	8.00	7.94	0.06	0.01	0.003

	It	***It/FS***
Infiltration Rate (inch/hr)*:	**0.01**	**0.003**

Reference: City of Santee BMP Design Manual (2016)
*Based on the drop measured in the final reading.

PERCOLATION TEST DATA

Project No.: 160392.2
Project Name: Mast Park Improvements
Test Date: June 30, 2016
Test Boring No.: **IF-4**
Diameter of Boring (D): 8.0 inch
Depth of Boring (d_b): 36.0 inch
Performed by: SM



Sandy Soil Criteria Test

Time of Testing			Water Level Measurements			Greater than or Equal to 6"?
Start Time T_i	Stop Time T_f	Time Interval (min) ΔT	Initial depth to water (inch) d_1	Final depth to water (inch) d_2	Drop of water column (inch) $\Delta d = d_i - d_f$	(Yes/No)
11:40 AM	12:10 PM	30	24.00	31.00	7.00	Yes
12:12 PM	12:42 PM	30	24.00	30.50	6.50	Yes

Factor of Safety (FS) = 4.375

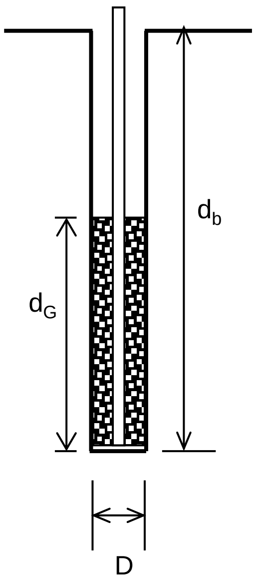
Time of Testing			Water Level Measurements		Water Level Calculations			Infiltration Rate Calculations	
Start Time T_i	Stop Time T_f	Time Interval ΔT (min)	Initial depth to water d_1 (inch)	Final depth to water d_2 (inch)	Initial height of water column d_i (inch)	Final height of water column d_f (inch)	Drop of water column $\Delta d = d_i - d_f$ (inches)	Tested Infiltration Rate ***It*** (inch/hr)	Design Infiltration Rate ***It/FS*** (inch/hr)
Percolation Test									
12:47 PM	12:57 PM	10	24.00	30.00	12.00	6.00	6.00	6.55	1.50
12:59 PM	1:09 PM	10	24.00	29.00	12.00	7.00	5.00	5.22	1.19
1:11 PM	1:21 PM	10	24.00	28.50	12.00	7.50	4.50	4.60	1.05
1:23 PM	1:33 PM	10	24.00	28.50	12.00	7.50	4.50	4.60	1.05
1:35 PM	1:45 PM	10	24.00	27.75	12.00	8.25	3.75	3.71	0.85
1:46 PM	1:56 PM	10	24.00	27.50	12.00	8.50	3.50	3.43	0.78
1:58 PM	2:08 PM	10	24.00	27.38	12.00	8.63	3.38	3.29	0.75
2:13 PM	2:23 PM	10	24.00	27.25	12.00	8.75	3.25	3.15	0.72
								It	***It/FS***
							Infiltration Rate (inch/hr)*:	**3.15**	**0.72**

Reference: City of Santee BMP Design Manual (2016)

*Based on the drop measured in the final reading.

PERCOLATION TEST DATA

Project No.: 160392.2
Project Name: Mast Park Improvements
Test Date: June 30, 2016
Test Boring No.: **IF-6**
Diameter of Boring (D): 8.0 inch
Depth of Boring (d_b): 36.0 inch
Performed by: SM



Sandy Soil Criteria Test

Time of Testing			Water Level Measurements			Greater than or Equal to 6"?
Start Time T_i	Stop Time T_f	Time Interval (min) ΔT	Initial depth to water (inch) d_1	Final depth to water (inch) d_2	Drop of water column (inch) $\Delta d = d_i - d_f$	(Yes/No)
11:02 AM	11:32 AM	30	28.00	32.50	4.50	No
11:35 AM	12:05 PM	30	28.00	32.00	4.00	No

Factor of Safety (FS) = 4.375

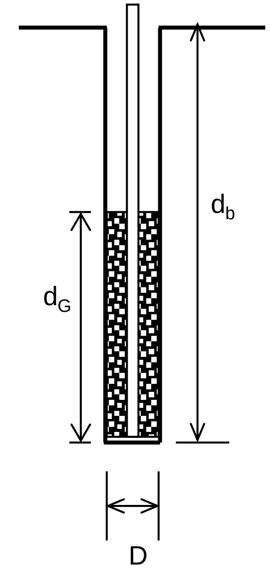
Time of Testing			Water Level Measurements		Water Level Calculations			Infiltration Rate Calculations	
Start Time T_i	Stop Time T_f	Time Interval ΔT (min)	Initial depth to water d_1 (inch)	Final depth to water d_2 (inch)	Initial height of water column d_i (inch)	Final height of water column d_f (inch)	Drop of water column $\Delta d = d_i - d_f$ (inches)	Tested Infiltration Rate ***It*** (inch/hr)	Design Infiltration Rate ***It/FS*** (inch/hr)
Percolation Test									
12:13 PM	12:43 PM	30	28.00	31.50	8.00	4.50	3.50	1.70	0.39
12:44 PM	1:14 PM	30	28.00	31.00	8.00	5.00	3.00	1.41	0.32
1:15 PM	1:45 PM	30	28.00	30.50	8.00	5.50	2.50	1.14	0.26
1:46 PM	2:18 PM	32	28.00	30.50	8.00	5.50	2.50	1.07	0.24
2:20 PM	2:51 PM	31	28.00	30.38	8.00	5.63	2.38	1.04	0.24
2:52 PM	3:23 PM	31	28.00	30.31	8.00	5.69	2.31	1.01	0.23
3:23 PM	3:54 PM	31	28.00	30.25	8.00	5.75	2.25	0.98	0.22

	It	***It/FS***
Infiltration Rate (inch/hr)*:	**0.98**	**0.22**

Reference: City of Santee BMP Design Manual (2016)

*Based on the drop measured in the final reading.

PERCOLATION TEST DATA



Project No.: 160392.2
Project Name: Mast Park Improvements
Test Date: June 30, 2016
Test Boring No.: **IF-7**
Diameter of Boring (D): 8.0 inch
Depth of Boring (d_b): 36.0 inch
Performed by: SM

Sandy Soil Criteria Test

Time of Testing			Water Level Measurements			Greater than or Equal to 6"?
Start Time T_i	Stop Time T_f	Time Interval (min) ΔT	Initial depth to water (inch) d_1	Final depth to water (inch) d_2	Drop of water column (inch) $\Delta d = d_i - d_f$	(Yes/No)
10:30 AM	11:00 AM	30	28.00	36.00	8.00	Yes
11:05 AM	11:35 AM	30	28.00	36.00	8.00	Yes

Factor of Safety (FS) = 4.375

Time of Testing			Water Level Measurements		Water Level Calculations			Infiltration Rate Calculations	
Start Time T_i	Stop Time T_f	Time Interval ΔT (min)	Initial depth to water d_1 (inch)	Final depth to water d_2 (inch)	Initial height of water column d_i (inch)	Final height of water column d_f (inch)	Drop of water column $\Delta d = d_i - d_f$ (inches)	Tested Infiltration Rate ***It*** (inch/hr)	Design Infiltration Rate ***It/FS*** (inch/hr)
Percolation Test									
11:10:00 AM	11:12:00 AM	2.00	30.00	36.00	6.00	0.00	6.00	72.00	16.46
11:13:00 AM	11:15:30 AM	2.50	30.00	36.00	6.00	0.00	6.00	57.60	13.17
11:17:00 AM	11:20:00 AM	3.00	30.00	36.00	6.00	0.00	6.00	48.00	10.97
11:22:00 AM	11:25:30 AM	3.50	30.00	36.00	6.00	0.00	6.00	41.14	9.40
11:27:00 AM	11:30:30 AM	3.50	30.00	36.00	6.00	0.00	6.00	41.14	9.40
11:33:00 AM	11:36:30 AM	3.50	30.00	36.00	6.00	0.00	6.00	41.14	9.40
11:38:00 AM	11:41:45 AM	3.75	30.00	36.00	6.00	0.00	6.00	38.40	8.78
11:43:00 AM	11:46:45 AM	3.75	30.00	36.00	6.00	0.00	6.00	38.40	8.78
11:49:00 AM	11:52:45 AM	3.75	30.00	36.00	6.00	0.00	6.00	38.40	8.78
11:55:00 AM	11:59:00 AM	4.00	30.00	36.00	6.00	0.00	6.00	36.00	8.23
12:01:00 PM	12:05:00 PM	4.00	30.00	36.00	6.00	0.00	6.00	36.00	8.23
12:06:00 PM	12:10:45 PM	4.75	30.00	36.00	6.00	0.00	6.00	30.32	6.93
12:12:00 PM	12:16:45 PM	4.75	30.00	36.00	6.00	0.00	6.00	30.32	6.93
12:18:00 PM	12:22:45 PM	4.75	30.00	36.00	6.00	0.00	6.00	30.32	6.93
12:24:00 PM	12:28:45 PM	4.75	30.00	36.00	6.00	0.00	6.00	30.32	6.93
								It	***It/FS***
							Infiltration Rate (inch/hr)*:	**30.32**	**6.93**

Reference: City of Santee BMP Design Manual (2016)
*Based on the drop measured in the final reading.

Worksheet 0-1: Factor of Safety and Design Infiltration Rate Worksheet

Factor of Safety and Design Infiltration Rate Worksheet			Worksheet D.5-1		
Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
A	Suitability Assessment	Soil assessment methods	0.25	1	0.25
		Predominant soil texture	0.25	3	0.75
		Site soil variability	0.25	3	0.75
		Depth to groundwater / impervious layer	0.25	3	0.75
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			2.5
B	Design	Level of pretreatment/ expected sediment loads	0.5	2*	1.00
		Redundancy/resiliency	0.25	2*	0.50
		Compaction during construction	0.25	1*	0.25
		Design Safety Factor, $S_B = \Sigma p$			1.75
Combined Safety Factor, $S_{total} = S_A \times S_B$				4.375* (* may be revised)	
Observed Infiltration Rate, inch/hr, $K_{observed}$ (corrected for test-specific bias)					
Design Infiltration Rate, in/hr, $K_{design} = K_{observed} / S_{total}$					
Supporting Data					
Briefly describe infiltration test and provide reference to test forms:					

Worksheet 0-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition		Worksheet C.4-1	
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	Yes, for specific locations	
Provide basis: Only locations IF-2, IF-4, and IF-7 have infiltration rates above 0.5 in/hr. Since the soil profile at the site is highly variable, additional testing is recommended for these locations prior to construction. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		No
Provide basis: Based on our review of the site topographic map, no slopes greater than 25% are present except for the river bank area. Site is located within a mapped liquefaction zone and the liquefaction potential at the site is high. Since the groundwater level is shallow, infiltration is not anticipated to significantly increase the liquefaction potential at the site. Groundwater mounding is anticipated due to the variable fine content of site soils. BMPs should be located away from existing utilities which we estimate are in the vicinity of the restroom building area. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>No</td></tr>
<tr><td colspan="4">Provide basis:

Groundwater was encountered at shallow depths ranging from 6 to 9 feet below existing grades at the site. The minimum recommended 10 ft distance between the bottom of the BMP and the ground water level is not available to partially remove contamination.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:

This question should be responded by a hydrologist.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">If all answers to rows 1 - 4 are “Yes” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration

If any answer from row 1-4 is “No”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2</td><td></td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by [City Engineer] to substantiate findings.

Worksheet C.4-1 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	Yes, for specific locations	

Provide basis:

Only locations IF-2, IF-4, and IF-7 have infiltration rates above 0.5 in/hr. Since the soil profile at the site is highly variable, additional testing is recommended for these locations prior to construction.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	Yes	No
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	Yes, for specific hazards	

Provide basis:

Based on our review of the site topographic map, no slopes greater than 25% are present except for the river bank area.
Site is located within a mapped liquefaction zone and the liquefaction potential at the site is high. Since the groundwater level is shallow, infiltration is not anticipated to significantly increase the liquefaction potential at the site.
Groundwater mounding is anticipated due to the variable fine content of site soils.
BMPs should be located away from existing utilities which we estimate are in the vicinity of the restroom building area.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>No</td></tr>
<tr><td colspan="4">Provide basis:

Groundwater was encountered at shallow depths ranging from 6 to 9 feet below existing grades at the site. The minimum recommended 10 ft distance between the bottom of the BMP and the ground water level is not available to partially remove contamination.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:

This question should be responded by a hydrologist.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.

If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No infitration</td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings

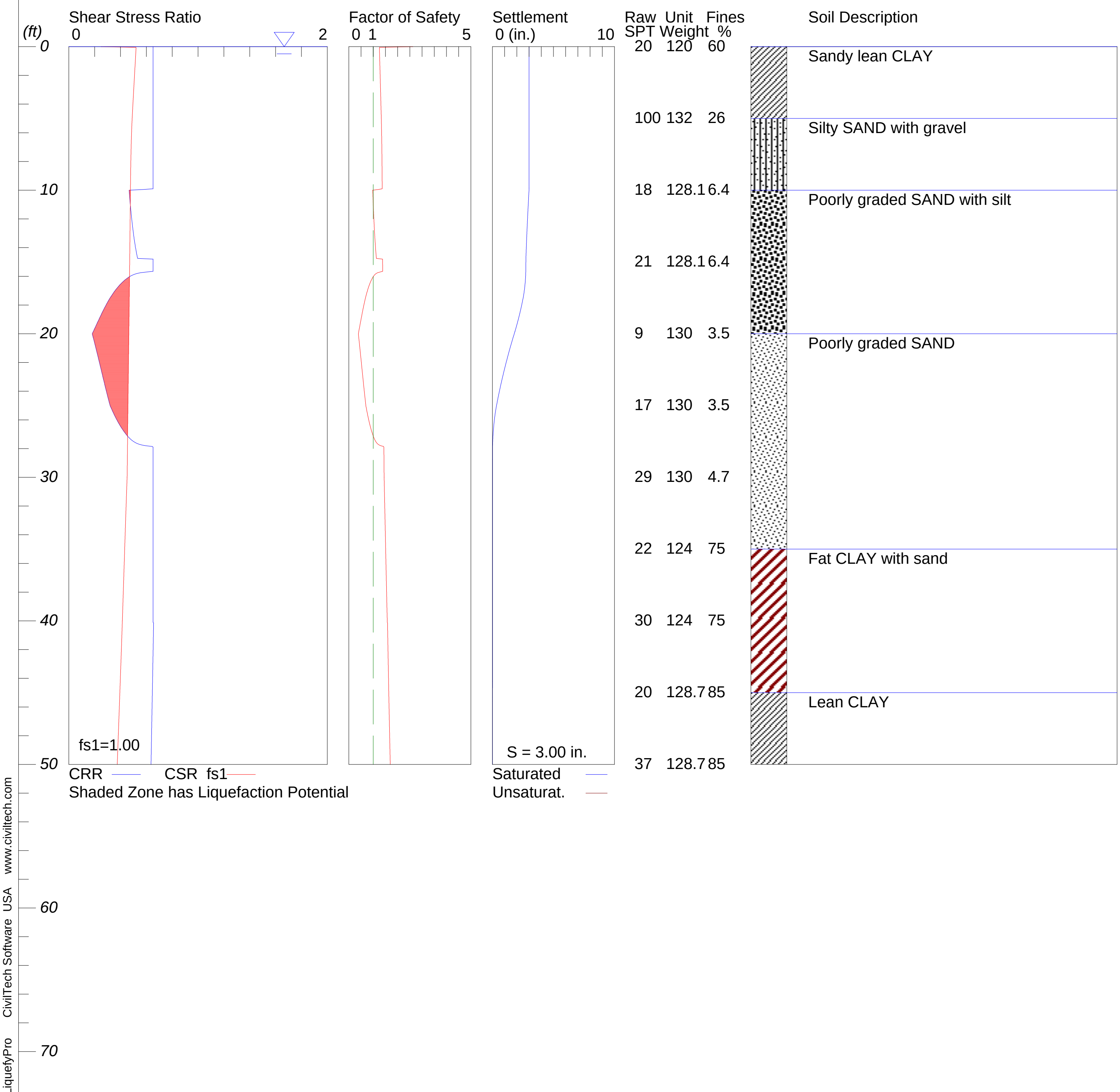
APPENDIX D
LIQUEFACTION ANALYSES

LIQUEFACTION ANALYSIS

Mast Park Improvement Project

Hole No.=B-1   Water Depth=0 ft   Surface Elev.=325

Magnitude=6.76
Acceleration=0.385g



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LIQUEFACTION ANALYSIS SUMMARY
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Licensed to , 8/31/2016 6:18:42 PM

Input File Name: M:\Projects\2016 Projects\160392.2- Task Order #16- Mast Park Improvement Project\Appendix D- Liquefaction\mast park- 1.liq
Title: Mast Park Improvement Project
Subtitle: Santee, California
Surface Elev.=325
Hole No.=B-1
Depth of Hole= 50.00 ft
Water Table during Earthquake= 0.00 ft
Water Table during In-Situ Testing= 10.00 ft
Max. Acceleration= 0.38 g
Earthquake Magnitude= 6.76

Input Data:
Surface Elev.=325
Hole No.=B-1
Depth of Hole=50.00 ft
Water Table during Earthquake= 0.00 ft
Water Table during In-Situ Testing= 10.00 ft
Max. Acceleration=0.38 g
Earthquake Magnitude=6.76
No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1
Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	20.00	120.00	60.00
5.00	100.00	132.00	26.00
10.00	18.00	128.10	6.40
15.00	21.00	128.10	6.40
20.00	9.00	130.00	3.50
25.00	17.00	130.00	3.50
30.00	29.00	130.00	4.70
35.00	22.00	124.00	75.00
40.00	30.00	124.00	75.00
45.00	20.00	128.70	85.00
50.00	37.00	128.70	85.00

Output Results:
Settlement of Saturated Sands=3.00 in.
Settlement of Unsaturated Sands=0.00 in.
Total Settlement of Saturated and Unsaturated Sands=3.00 in.
Differential Settlement=1.501 to 1.981 in.

Depth	CRRm	CSRfs	F.S.	S_sat.	S_dry	S_all

ft				in.	in.	in.
0.00	0.65	0.25	2.61	3.00	0.00	3.00
1.00	0.65	0.51	1.27	3.00	0.00	3.00
2.00	0.65	0.51	1.28	3.00	0.00	3.00
3.00	0.65	0.50	1.30	3.00	0.00	3.00
4.00	0.65	0.50	1.31	3.00	0.00	3.00
5.00	0.65	0.49	1.33	3.00	0.00	3.00
6.00	0.65	0.49	1.34	3.00	0.00	3.00
7.00	0.65	0.48	1.35	3.00	0.00	3.00
8.00	0.65	0.48	1.36	3.00	0.00	3.00
9.00	0.65	0.48	1.36	3.00	0.00	3.00
10.00	0.47	0.48	0.98*	3.00	0.00	3.00
11.00	0.48	0.48	1.00	2.93	0.00	2.93
12.00	0.49	0.47	1.03	2.88	0.00	2.88
13.00	0.50	0.47	1.06	2.82	0.00	2.82
14.00	0.52	0.47	1.09	2.77	0.00	2.77
15.00	0.65	0.47	1.38	2.74	0.00	2.74
16.00	0.47	0.47	1.01	2.72	0.00	2.72
17.00	0.36	0.47	0.76*	2.61	0.00	2.61
18.00	0.29	0.47	0.62*	2.40	0.00	2.40
19.00	0.23	0.47	0.50*	2.12	0.00	2.12
20.00	0.18	0.46	0.39*	1.79	0.00	1.79
21.00	0.21	0.46	0.45*	1.44	0.00	1.44
22.00	0.24	0.46	0.51*	1.13	0.00	1.13
23.00	0.26	0.46	0.57*	0.84	0.00	0.84
24.00	0.29	0.46	0.63*	0.57	0.00	0.57
25.00	0.32	0.46	0.70*	0.34	0.00	0.34
26.00	0.37	0.46	0.81*	0.16	0.00	0.16
27.00	0.44	0.46	0.97*	0.05	0.00	0.05
28.00	0.65	0.45	1.44	0.01	0.00	0.01
29.00	0.65	0.45	1.44	0.01	0.00	0.01
30.00	0.65	0.45	1.44	0.01	0.00	0.01
31.00	0.65	0.45	1.46	0.01	0.00	0.01
32.00	0.65	0.44	1.47	0.01	0.00	0.01
33.00	0.65	0.44	1.48	0.01	0.00	0.01
34.00	0.65	0.44	1.50	0.01	0.00	0.01
35.00	0.65	0.43	1.51	0.01	0.00	0.01
36.00	0.65	0.43	1.52	0.01	0.00	0.01
37.00	0.65	0.43	1.53	0.01	0.00	0.01
38.00	0.65	0.42	1.55	0.01	0.00	0.01
39.00	0.65	0.42	1.56	0.01	0.00	0.01
40.00	0.65	0.41	1.57	0.01	0.00	0.01
41.00	0.65	0.41	1.59	0.01	0.00	0.01
42.00	0.65	0.41	1.60	0.01	0.00	0.01
43.00	0.65	0.40	1.61	0.01	0.00	0.01
44.00	0.65	0.40	1.63	0.01	0.00	0.01
45.00	0.65	0.39	1.64	0.00	0.00	0.00
46.00	0.64	0.39	1.65	0.00	0.00	0.00
47.00	0.64	0.39	1.66	0.00	0.00	0.00
48.00	0.64	0.38	1.67	0.00	0.00	0.00
49.00	0.64	0.38	1.68	0.00	0.00	0.00
50.00	0.64	0.37	1.70	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)

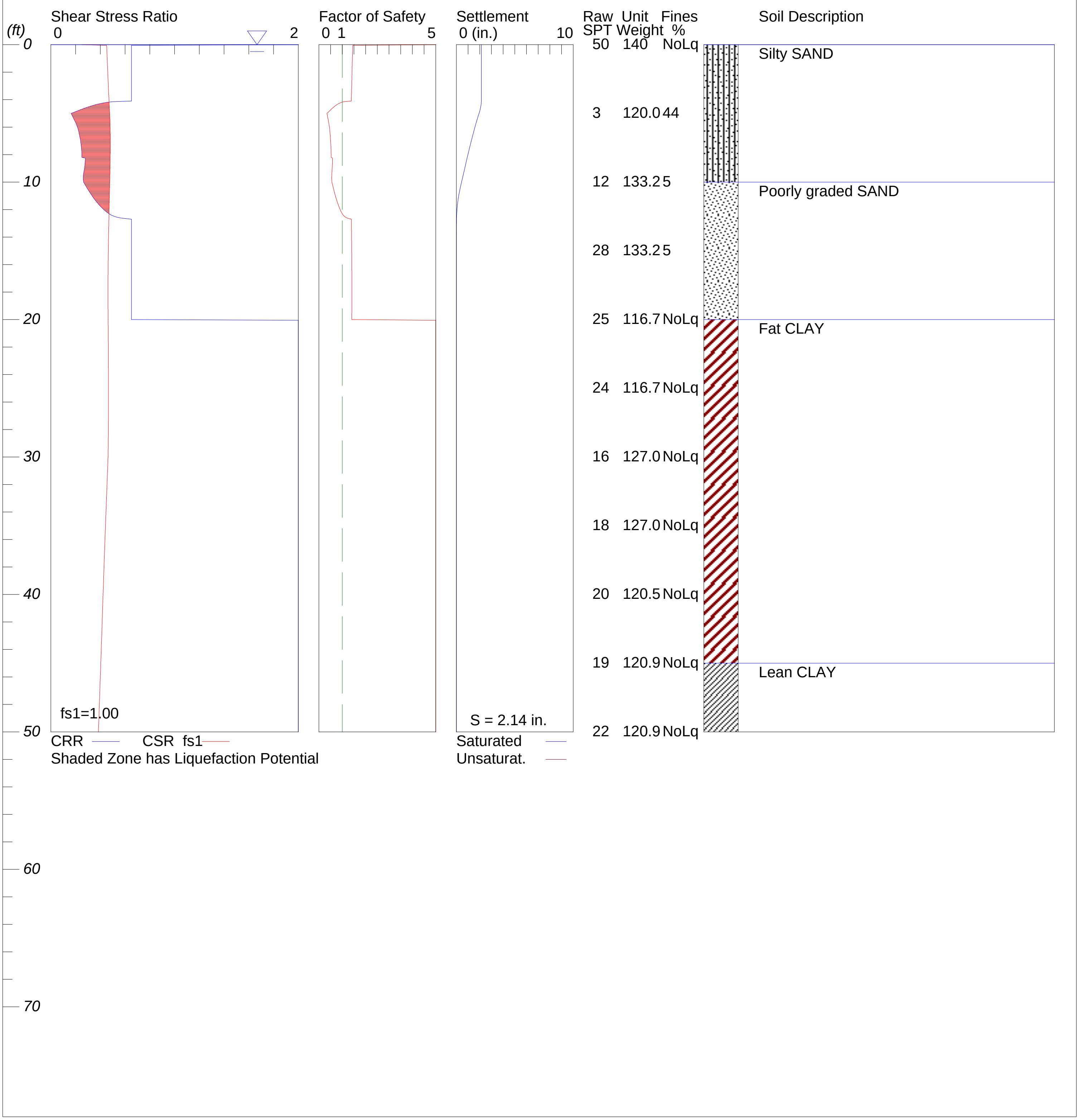
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

Mast Park Improvement Project

Hole No.=B-2 Water Depth=0 ft Surface Elev.=322

Magnitude=6.76
Acceleration=0.385g



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LIQUEFACTION ANALYSIS SUMMARY
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Input File Name: M:\Projects\2016 Projects\160392.2- Task Order #16- Mast Park Improvement Project\Appendix D- Liquefaction\mast park- 2.liq
Title: Mast Park Improvement Project
Subtitle: Santee, California
Surface Elev.=322
Hole No.=B-2
Depth of Hole= 50.00 ft
Water Table during Earthquake= 0.00 ft
Water Table during In-Situ Testing= 10.00 ft
Max. Acceleration= 0.38 g
Earthquake Magnitude= 6.76

Input Data:
Surface Elev.=322
Hole No.=B-2
Depth of Hole=50.00 ft
Water Table during Earthquake= 0.00 ft
Water Table during In-Situ Testing= 10.00 ft
Max. Acceleration=0.38 g
Earthquake Magnitude=6.76
No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1
Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	50.00	140.00	NoLiq
5.00	3.00	120.00	44.00
10.00	12.00	133.20	5.00
15.00	28.00	133.20	5.00
20.00	25.00	116.70	NoLiq
25.00	24.00	116.70	NoLiq
30.00	16.00	127.00	NoLiq
35.00	18.00	127.00	NoLiq
40.00	20.00	120.50	NoLiq
45.00	19.00	120.90	NoLiq
50.00	22.00	120.90	NoLiq

Output Results:
Settlement of Saturated Sands=2.14 in.
Settlement of Unsaturated Sands=0.00 in.
Total Settlement of Saturated and Unsaturated Sands=2.14 in.
Differential Settlement=1.068 to 1.410 in.

Depth CRRm CSRfs F.S. S_sat. S_dry S_all

ft				in.	in.	in.
0.00	2.00	0.25	5.00	2.14	0.00	2.14
1.00	0.65	0.46	1.43	2.14	0.00	2.14
2.00	0.65	0.46	1.42	2.14	0.00	2.14
3.00	0.65	0.46	1.40	2.14	0.00	2.14
4.00	0.65	0.47	1.39	2.14	0.00	2.14
5.00	0.16	0.48	0.34*	1.94	0.00	1.94
6.00	0.22	0.48	0.45*	1.59	0.00	1.59
7.00	0.24	0.48	0.50*	1.28	0.00	1.28
8.00	0.25	0.48	0.52*	0.99	0.00	0.99
9.00	0.27	0.48	0.57*	0.72	0.00	0.72
10.00	0.26	0.48	0.56*	0.44	0.00	0.44
11.00	0.33	0.47	0.71*	0.20	0.00	0.20
12.00	0.43	0.47	0.91*	0.05	0.00	0.05
13.00	0.65	0.47	1.39	0.01	0.00	0.01
14.00	0.65	0.47	1.40	0.00	0.00	0.00
15.00	0.65	0.46	1.40	0.00	0.00	0.00
16.00	0.65	0.46	1.41	0.00	0.00	0.00
17.00	0.65	0.46	1.41	0.00	0.00	0.00
18.00	0.65	0.46	1.41	0.00	0.00	0.00
19.00	0.65	0.46	1.41	0.00	0.00	0.00
20.00	0.65	0.46	1.41	0.00	0.00	0.00
21.00	2.00	0.46	5.00	0.00	0.00	0.00
22.00	2.00	0.46	5.00	0.00	0.00	0.00
23.00	2.00	0.47	5.00	0.00	0.00	0.00
24.00	2.00	0.47	5.00	0.00	0.00	0.00
25.00	2.00	0.47	5.00	0.00	0.00	0.00
26.00	2.00	0.47	5.00	0.00	0.00	0.00
27.00	2.00	0.47	5.00	0.00	0.00	0.00
28.00	2.00	0.46	5.00	0.00	0.00	0.00
29.00	2.00	0.46	5.00	0.00	0.00	0.00
30.00	2.00	0.46	5.00	0.00	0.00	0.00
31.00	2.00	0.46	5.00	0.00	0.00	0.00
32.00	2.00	0.45	5.00	0.00	0.00	0.00
33.00	2.00	0.45	5.00	0.00	0.00	0.00
34.00	2.00	0.45	5.00	0.00	0.00	0.00
35.00	2.00	0.44	5.00	0.00	0.00	0.00
36.00	2.00	0.44	5.00	0.00	0.00	0.00
37.00	2.00	0.43	5.00	0.00	0.00	0.00
38.00	2.00	0.43	5.00	0.00	0.00	0.00
39.00	2.00	0.43	5.00	0.00	0.00	0.00
40.00	2.00	0.42	5.00	0.00	0.00	0.00
41.00	2.00	0.42	5.00	0.00	0.00	0.00
42.00	2.00	0.41	5.00	0.00	0.00	0.00
43.00	2.00	0.41	5.00	0.00	0.00	0.00
44.00	2.00	0.41	5.00	0.00	0.00	0.00
45.00	2.00	0.40	5.00	0.00	0.00	0.00
46.00	2.00	0.40	5.00	0.00	0.00	0.00
47.00	2.00	0.40	5.00	0.00	0.00	0.00
48.00	2.00	0.39	5.00	0.00	0.00	0.00
49.00	2.00	0.39	5.00	0.00	0.00	0.00
50.00	2.00	0.38	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

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F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
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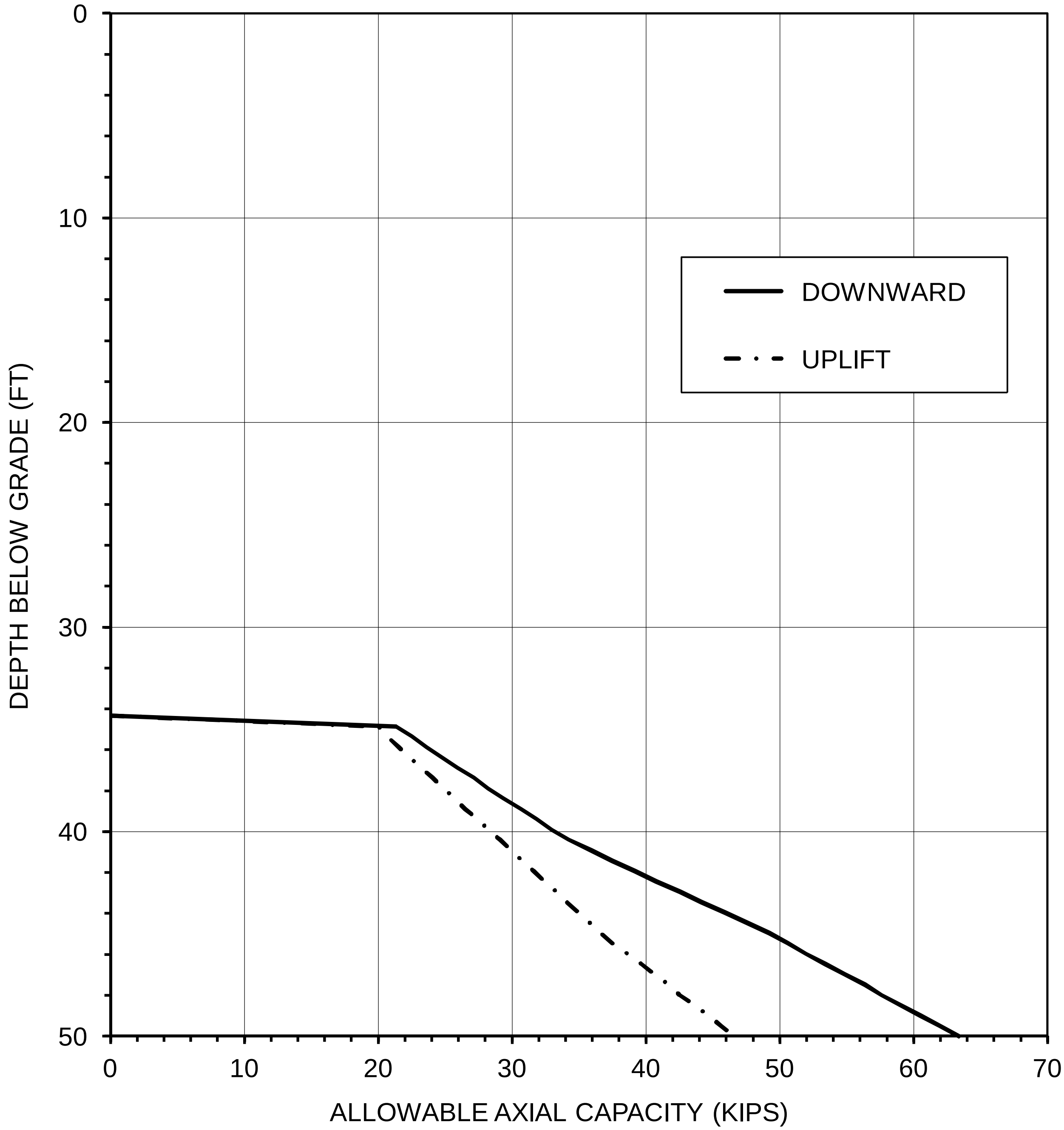
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

APPENDIX E
PILE CAPACITY ANALYSES

14-INCH DRIVEN PRECAST SQUARE CONCRETE PILES



TWINING

AXIAL PILE CAPACITY CURVES		
MAST PARK IMPROVEMENT PROJECT 9125 CARLTON HILLS BLVD SANTEE, CALIFORNIA		
PROJECT NO. 160392.2	DATE August 2016	FIGURE E-1